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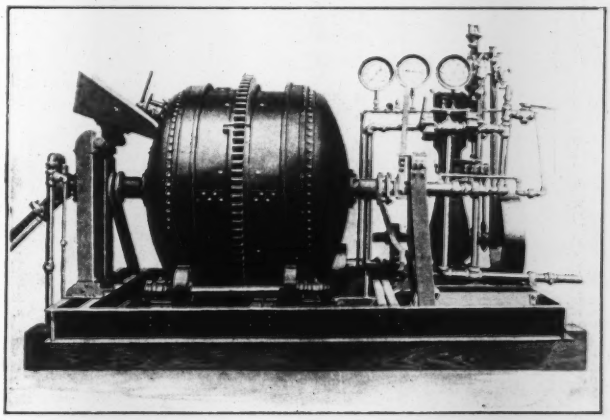
The New Pneumatic Mixer

The Three-in-One Process of Mixing, Conveying and Placing

Now meets all conditions of concrete work



Four years' experience and cost keeping has conclusively demonstrated the great economy in the 3-in-1 Pneumatic mixing—conveying—placing process.

The new Pneumatic Mixer, recently brought out, makes these savings still greater. It brings Pneumatic efficiency to every job, large or small. On **any** job the following is the gang now required:

- **2** men feeding cement, sand and rock
- **1** operator
- **1** man placing concrete
- **1** man's time applied to shifting pipe (part time on some jobs)

In addition—the new machine cuts compressed air consumption in half over old type consumption.

This new economy is yours because we have solved the problem of shooting 1½-in. rock or gravel concrete through a 3-in. hose and 3-in. pipe.

It can't clog!

Made in three sizes:

B2—Two Bag Batch—3- or 4-in. outlet pipe as desired—capacity with 3-in. pipe, 15-20 cu.yds. per hr. For buildings, roads, culverts, abutments and retaining walls.

B3—Three Bag Batch—4x6-in. pipe—40 to 50 cu.yd. per hour. For tunnel work large and small, retaining walls, track elevation and all standard railroad work, heavy foundation and bridge work.

B4—Four Bag Batch—6-in. to 8-in. discharge. 60 to 80 cu.yds. per hour with 6-in. pipe. For dams, bridge piers and all heavy work.

We will furnish all particulars at your request or will demonstrate the new mixer on your own job.

Pneumatic Placing Co.
Incorporated
2 Rector Street
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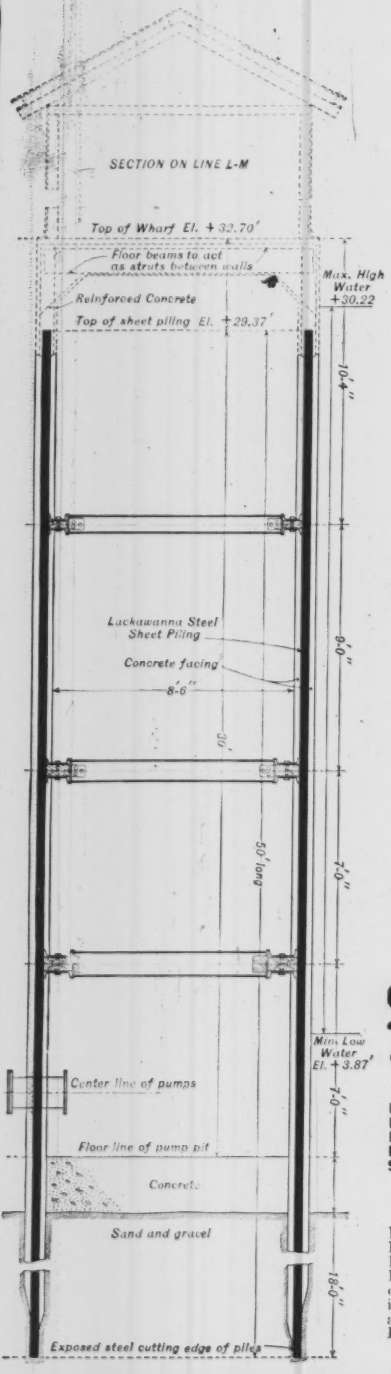
In Alaska, This Lackawanna Protected Steel Sheet Piling Structure Withstands A 26-Foot Tide



The pump house of the Alaska-Treadwell Gold Mining Company, Treadwell, Alaska, contains three pumps set over a pit 35 feet deep, and is located where the tide is over 26 feet, and where at low tide, the water is 9 feet deep.

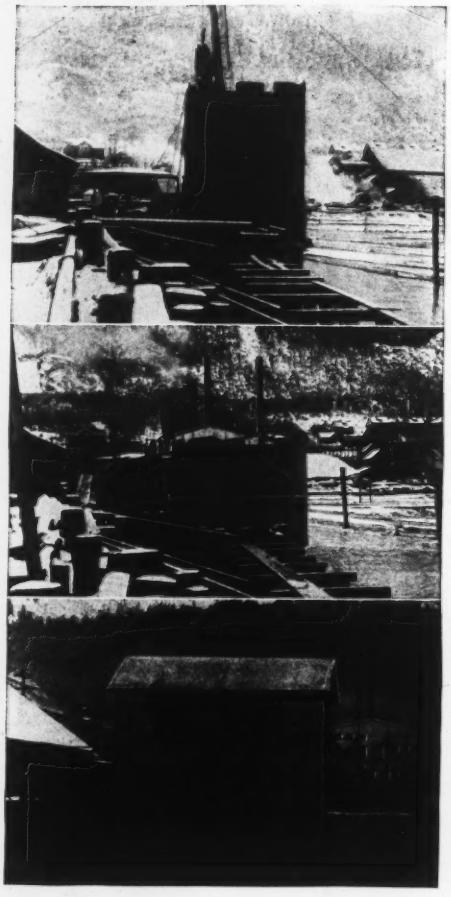
To assure a perfectly rigid structure under the unusual strain of a 26-foot tide the side walls were constructed of Lackawanna Arch-Web Sheet Piling, braced internally with struts and waling, and driven to 18-foot penetration in soil below the water. Watertightness was secured by a two-foot concrete floor at the bottom, by facing the steel sheet piling inside and out with concrete, and by pouring a concrete mixture into the interlocks of all sheet piling joints. The bond between concrete and steel sheet piling was strengthened by lacing small reinforcing bars through the piling before pouring the concrete.

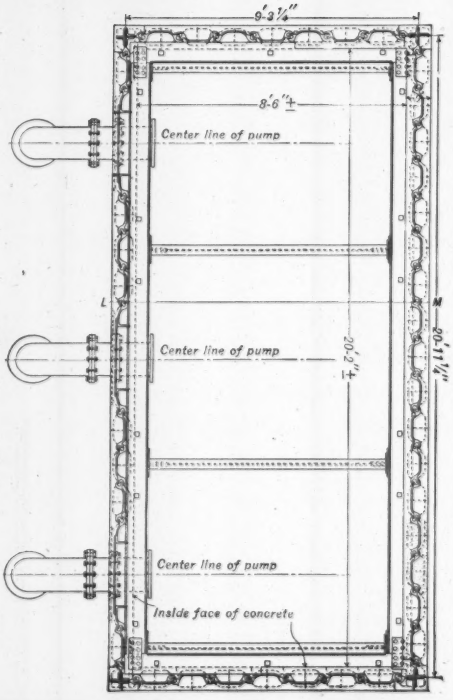
The steel sheet piling which was fabricated before shipment, was driven without difficulty and the finished structure is strong, neat in appearance and in every way up to expectations.

The pictures show this work, *first*, Piling assembled and while it was being driven, *second*, with forms in place for pouring the concrete and *third*, after completion.

The design was prepared by C. C. Moore & Co. of San Francisco, and the construction was carried out under the direction of Mr. P. R. Bradley, Construction Engineer of the Alaska-Treadwell Co.

Our book "Lackawanna Steel Sheet Piling" describes all common and many uncommon sheet piling uses. Ask for a copy, and also for the free advice of our Steel Sheet Piling Engineers.





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Engineering News

VOLUME 76 AUGUST 24, 1916 NUMBER 8

Popolopen Steel Arch in the Hudson Highlands

The new steel arch bridge over Popolopen Creek, 5 mi. south of West Point, N. Y., was opened to traffic by the Palisades Interstate Park Commission on Saturday, July 15. The competitive designs for this structure were published in *Engineering News* of Oct. 28, 1915, and the accompanying view shows the completed bridge, which was designed and built for the commission by the Phoenix Bridge Co. The piers for the bridge were constructed by the New York State Highway Department, under the arrangement noted further on.

STEEL ARCH BRIDGE OVER POPOLOPEN CREEK BETWEEN BEAR MOUNTAIN AND WEST POINT, N. Y.

Popolopen Creek is a narrow gorge with steep rocky walls nearly 150 ft. high, extending back from the shore of the Hudson about half a mile. The only road in the valley of the Hudson on this side of the river has hitherto crossed the creek by making a sharp turn at the rim of the gorge, descending into the gorge by a narrow and poorly guarded roadway on a very steep grade to a bridge near the water level and ascending by an equally steep road on the other side. The traditional name of the gorge is Hell Hole, and the frequency of accidents there, since the road carries a heavy motor traffic, explains the name.

From an engineering point of view the obvious thing to do was to build a high-level bridge straight across the gorge, but the New York Highway Department is forbidden by law to use its appropriation from the state in the construction of bridge superstructures larger than small culverts. The county, on the other hand, took comparatively little interest in the bridge, since the road of which it forms a part is chiefly of importance as a through north and south line down the Hudson Valley, and the population of the county is mostly west of the highlands that border the Hudson.

It happens that the Palisades Interstate Park Commission, however, under the broad powers granted to it in 1910, has authority to protect the scenic features of the Hudson Highlands and is working toward the completion of a through north and south road on the west side of the river. The Bear Mountain Park and pleasure grounds, under control of the commission, border Popolopen Creek on the south. The commission therefore arranged to furnish the funds for the superstructure of the bridge, and the New York Highway Department built the piers.

While the bridge is commonly spoken of as an arch, structural engineers will recognize that it is actually a cantilever structure. The floor of the bridge is of reinforced concrete, on which eventually an asphalt wearing surface is to be laid. At present, however, the traffic, which is chiefly automobiles, is being carried directly on the concrete, as it was considered that better results in the adhesion of the asphalt to the concrete would be obtained if the latter was subjected to wear for some months before the asphalt was applied.

By the abolition of the dangerous Hell Hole passage there is now a good and safe roadway for automobile traffic from New York City on the west side of the Hudson all the way to West Point. Work is in progress on the section of the road that is to run from West Point to Cornwall along the steep slope of Crow's Nest and Storm King Mountains. When this section is finished, there will be a continuous motor road along the west shore of the Hudson all the way between New York and Albany.

Constructing Pile Foundation in Hard Gravel

By Smith O. Steere*

Construction of the pier footings for the new Main St. bridge over the Allegheny River at Salamanca, N. Y., involved special difficulties in driving piles from a raft in a swift current into the hard stratum of gravel forming the river bed at the bridge site. Further, an unusual method of securing the foundation in varying heads of water was employed—the use of three-wall coffer-dams.

The 102 piles for each of the four river piers were driven by a piledriving plant installed on a raft. The plant consisted of an upright boiler, single-drum hoisting engine and a 2,000-lb. hammer with 24-ft. leads. The piles for each footing course were driven, beginning at the upstream end of the footing block, before any excavating was done and before the coffer-dams were built. The plans called for 24-ft. lengths of pile below cutoff, the elevation of cutoff being 1½ ft. above the bottom grade of the footing block, or on an average 3½ ft. beneath the river bed. So great was the resistance that the gravel offered to the piles that a penetration of only 20 to 22 ft. below stream bed could be attained for the average pile and then only after much constant pounding accompanied with considerable brooming and shearing off of some of the piles. At the start the progress of piledriving was slow. The penetration of the piles under the first blows of the hammer was not over 3 in. per blow. As the driving progressed the fall of the hammer increased, but the penetration per blow rapidly decreased until it was not over ½ in. for an 18-ft. fall of the hammer. Under these circumstances only about eight piles per day were driven, when the piles were used without protection, owing to the excessive brooming of the point.

In order to expedite the work a special pile shoe to protect the pile points was tried. The shoes proved effective. The penetration per blow of hammer was doubled, and the progress of the work was greatly increased. Steel straps for protecting the pile points were also experimented with, but without any appreciable success; they were soon abandoned.

Later on, while workmen were excavating for the footing courses, evidences of the efficiency of the pile shoes were found. Piles with the shoe attached had pierced large flat stones, stones that otherwise would have made the pile glance out of line. The accompanying photograph, Fig. 2, shows a pile shoe unearthed with the pile sheared off just above the point of attachment of the shoe to the pile after striking a large boulder.

The table gives the subcontractor's distribution of cost for driving 419 piles of 24-ft. length for the foundation courses of the bridge.

ITEMIZED COST OF PILE DRIVING

Freight charges to and from Salamanca	$74.40
Unloading and loading plant and team hire	41.50
Cost of barge (materials, labor and repairs)	81.85
Oil and grease	9.95
Bolts, nails and washers	6.32
Blacksmithing charges	27.65
Repairs to pile-driving plant	17.35
Rope for hammer	37.65
30 tons coal @ $4.50	135.00
Labor, July 27 to Nov. 9	799.35
Superintendence and overhead charges	300.00
Interest on plant investment for 3 mo.	9.00
Depreciation of plant for 3 mo.	15.00
	$1,555.02
Salvage on barge	40.00
Net cost, 419 piles	$1,515.02
Cost per lineal foot, $1,515.02 ÷ 10,056	.15

In order to carry the excavation for the footing courses to grade, three-wall coffer-dams were built of sheeting and bracing around each foundation site of the piers. The dimensions of the outer coffer-dam walls were 68x20 ft., and the spaces between walls were 2 ft.; inside of the inner wall the dimensions were 60x12 ft., or the same as the size of the footing block. The entire area within the inner sheetpile inclosure was filled with upstanding pile butts projecting 3 or 4 ft. above the river bed and 6 to 7 ft. above the saw-off elevation.

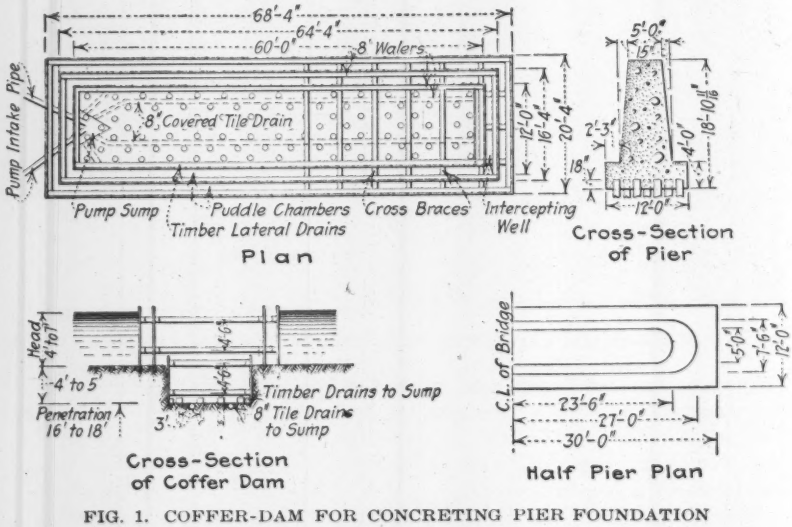


FIG. 1. COFFER-DAM FOR CONCRETING PIER FOUNDATION

The puddle chambers between the three walls of the coffers were filled with a clay-gravel puddle excavated at the south abutment site. The upper ends of all the coffer-dams showed considerable leakage, and these points were

*Former Assistant Engineer, New York Highway Commission, for Salamanca Bridge; Montela, Ulster County, New York.

strengthened and made more impervious to water by filling cement bags with clay and placing them around the outside.

After placing a coffer-dam, the interior was unwatered by using two electrically driven centrifugal pumps with 8-in. intake and 6-in. discharge pipes. These pumps were speeded up to a capacity of 700 gal. per min. each. Except where troublesome local leaks developed, the pumps easily handled the ordinary leakage through the

FIG. 2. PILE SHEARED OFF AFTER STRIKING BOULDER

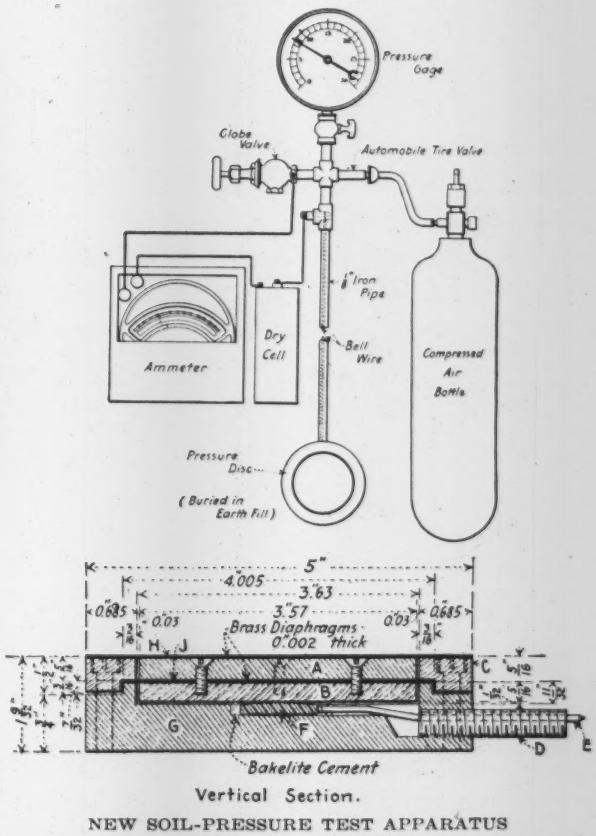
coffer walls and the seepage through the porous gravel of the river bed, which was considerable.

Workmen shoveled the material from the foundation pits by lift and wasted it over the edge of the coffer-dam. As the excavating proceeded downward, the sheeting and bracing of the inner coffer wall also were driven down, so that the bottom of the sheeting was kept 4 or 5 in. below the floor. The upstanding pile butts were sawed off near the river bed, as they interfered with the free use of the shovels; later on they were cut off at saw-off grade, 1½ ft. above the bottom grade of the footing block.

The two centrifugal pumps easily handled the water while excavation was going on, but the problem was how to keep the coffer-dam unwatered during the concreting of the foundation course, as the specifications prohibited the laying of concrete in still or running water. Finally, at the writer's suggestion, a plan was tried that proved satisfactory. A narrow intercepting well 2 ft. deep was dug across the upper end of the pit and fully planked. This intercepted the seepage and leakage at the upper end of the coffer, where it was the greatest, and delivered it through two lines of 8-in. tile drains laid under the floor to a triangular sump at the downstream end of the pit. Two additional intercepting side timber drains were placed just beneath the lower wales of the inner wall of the coffer-dam to take care of water coming through the sides; this also discharged into the pump sump. The sump had planked sides; its bottom was 2 ft. below the floor of the pit, and its top extended 2 ft. above the top grade of the foundation block. With this arrangement of intercepting well, tile and intercepting lateral drains and sump well, the concrete for the footing blocks was laid without any trouble whatsoever from water. When the concreting for the piers proper was done, the pump sump wells were unplanked and filled with concrete. Fig. 1 shows the plan and section of the drainage system.

Device To Test Soil Pressure

A. T. Goldbeck and T. B. Smith, of the United States Agricultural Department, have devised an instrument which it is hoped will accurately measure the pressure inside an earth-fill or even inside undisturbed original earth. The authors state that they have attempted to develop



NEW SOIL-PRESSURE TEST APPARATUS

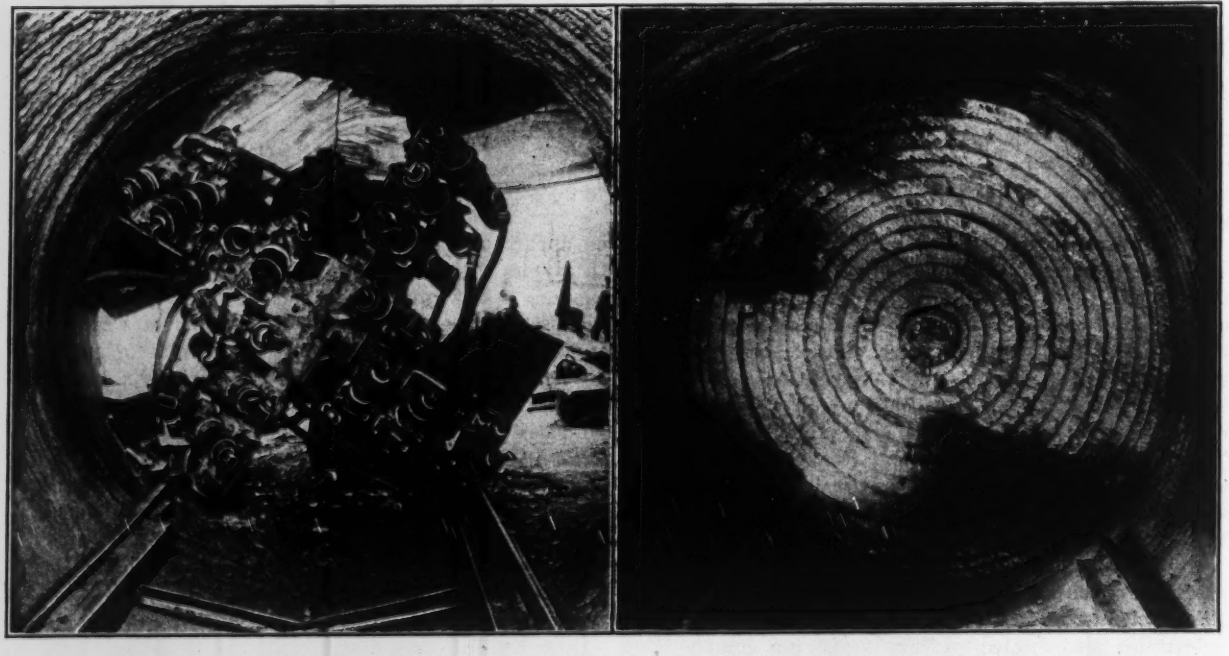
an apparatus that will measure such pressure with practically no disturbance of the natural condition of the ground, and in considering it it was deemed advisable to develop the idea of a portable cell of small size capable of indicating at some remote station the value of any pressure coming upon it. The device consists of a small diaphragm cell and a slightly movable top, the movement of which is indicated by electrical contact communicated to the outside of the fill through a wire leading from the inside of the cell through an armored pipe. This armored pipe also conducts compressed air from an outside tank to the interior of the cell, and the pressure of the soil is measured by the reaction of the disk movement against this air pressure. The cell is in some way similar to that used by Professor McDaniel in measuring the pressure of concrete in forms, described in *Engineering News*, May 18, 1916, p. 933.

Rock-Tunneling Machine Using Pneumatic-Hammer Tools

A very interesting rock-tunneling machine is at present doing demonstration work in the subway excavation at the Grand Central Terminal, New York City. This machine, which is a recent development from the old Terry, Tench & Proctor machine described in *Engineering News* of Nov. 19, 1908, is designed to chip away the rock by the action of a number of pneumatic hammers fitted with chisels which turn on a longitudinal axis.

The secret of the greater measure of success attained by this machine, which has been developed by Oliver O. App, under the name of the Rock Tunneling Machine Co., 79 East 130th St., New York City, is contained in the fact that there is a distinct relation between the character of the cutting and the action of the hammers. When a chisel is not bearing against the rock, there is no action of the hammer. As soon as the tool contacts firmly with the rock face the hammer operates. Until this coöperation with chisel and hammer was obtained, the hammer type of tunneling machine was bound to remain an experiment.

FIG. 1. GENERAL VIEW OF THE APP TUNNELER STARTING HEADING, GRAND CENTRAL TERMINAL, NEW YORK CITY



FIGS. 2 AND 3. VIEW OF CUTTING HEAD OF THE IMPROVED TUNNELER AND THE ROCK FACE

The carriage of the machine consists of a two-wheel main truck at the rear and a two-wheel guide truck at the front. The wheels rest on a light track of standard gage, shown in Figs. 1 and 2. The front and rear halves are separate; the front, carrying the revolving head, is moved forward 1 in. per revolution, by means of two lead screws. The hydraulic feed of the earlier machine has been abandoned. After the front has advanced 2 to 3 ft., the rear is moved up by means of gearing. The main shaft, which carries and rotates the head, is 12 in. in diameter and 14 ft. long and extends the full length of the machine along its axis.

The head, which is the tool carrier, comprises a hub and four arms placed 90° apart. Two arms carry four hammers each; on each of the other two arms are three hammers. The 14 hammers are so arranged that the chisels cut in concentric circles, producing the face illustrated in Fig. 3.

The power to rotate the head and to manipulate the machine is furnished by a compressed-air engine located on the forward part of the carriage.

When it becomes necessary to move the rear half of the machine forward, the track on which it rests is moved bodily by means of jacks. This method will be improved in future machines.

The Head Carries Fourteen Hammers

The hammer-and-chisel element has been entirely redesigned. In the old Terry-Proctor machine a 3-in. cylinder, with piston and valves and springs at each end of the

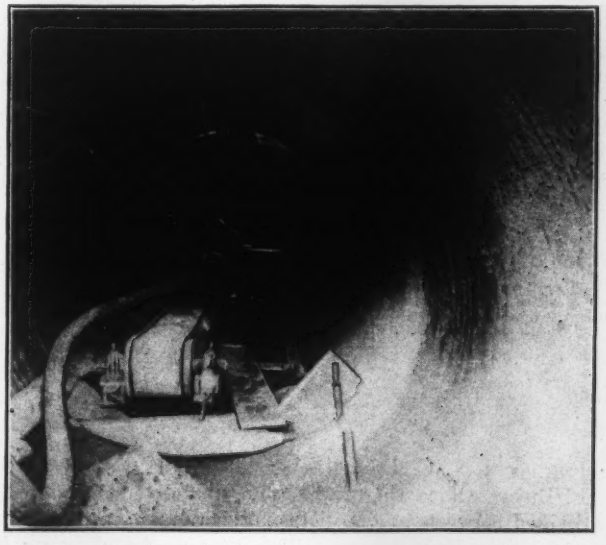


FIG. 4. REAR VIEW OF MACHINE IN TUNNEL, SHOWING AIR LINE AND SPOIL CONVEYOR

cylinder, was tried. Mr. App uses a 4-in. valveless cylinder carrying a 32-lb. hammer 12 in. long. The hammer is reciprocated by the action of compressed air, the distribution of which is effected entirely by the passing of the hammer over ports in the cylinder wall, there being no distributing valves whatever. The hammer strikes a steel chisel-holder, into which the chisel is firmly inserted. It is now proposed to form the tools with longer shanks so that the hammer will strike the tool direct and not the holder.

In the event of one or more of the tools meeting no resistance—an operating condition that may occur at any moment—the air supply to those particular cylinders is shut off immediately and automatically. Arrangements are provided so that the supply of air to any single hammer is determined solely by its immediate demand, and this control is effected by variation of bearing pressure of the tool against the rock. The harder the rock the more air is supplied automatically. If the rock is soft, less air is supplied; if sufficiently soft, the hammer is inactive and the rock is sheared off by the rotation of the

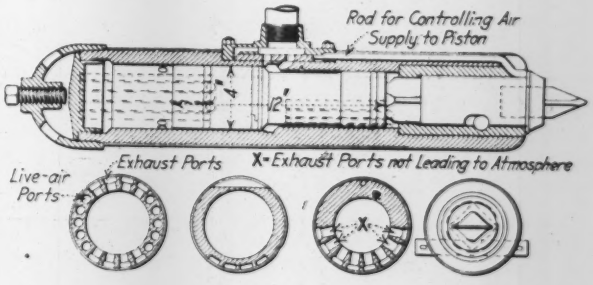


FIG. 5. LATEST DESIGN OF 32-LB. PNEUMATIC HAMMER

tool. In short, the blow delivered is proportional to the hardness of the rock to be cut.

Oil enters the cylinders with the air. The chisels now in use are made of 1½-in. square steel. The movement of a chisel is limited to not over ½ in.

Dust in the tunnel is practically eliminated by a spray of air and water over the face of the rock. The broken rock is elevated by two scoops carried by the rotating head and delivered on a belt conveyor (Fig. 4) that discharges into muck cars at the rear.

Record of Runs

In three 8-hr. shifts in which the total operating time was 13 hr., an advance was made of 8 ft. 10½ in. in the 8-ft. diameter tunnel. The lost time comprised that required to move track forward, to break off core and to insert tools that dropped out. The following is a record of an 8-hr. test and was compiled by H. S. Dow, of the Rapid Transit Construction Co., New York City:

TRIAL RUN ON JUNE 9, 1916

Operating from cut on north side 42nd St. between Park and Lexington Ave.

Time of Start, P.M.	Time of Stop, P.M.	Duration of Run, Min.	Cause of Stop	Time Machine Was Idle, Min.	Total Progress, Ft.	Total Progress, In.	Total Running Time, Hr.	Total Running Time, Min.
5:54	6:03	9	To break off rock ore..	13	..	..	..	..
6:16	6:27	11	To tighten tool	6	..	..	..	..
6:33	6:34	1	To change tools	13	..	..	..	..
6:47	6:53	6	To measure progress...	1	0	4	0	27
6:54	7:01	7	To change tools and move track	17	0	5	0	34
7:18	8:04	46	To change tools and move track	25	0	11	1	20
8:29	8:37	8	To clean out muck.....	4	..	..	..	..
8:41	8:46	5	To replace tools and move track	24	1	0	1	33
9:10	9:17	7	To replace tool; lunch..	54	1	2	1	40
10:11	10:21	10	To replace tool	3	..	..	..	..
10:24	10:30	6	To clean out muck.....	1	..	..	..	..
10:31	10:33	2	To break off rock core.	11	..	..	..	..
10:44	11:03	19	To clean out muck and replace tools	40	1	7	2	17
11:43 A.M.	12:01 A.M.	18	To tighten tools	3	..	..	..	..
12:04	12:11	7	To tighten tools	2	..	..	..	..
12:13	12:24	11	To tighten tools	3	..	..	..	..
12:27	12:35	8	To clean out muck.....	40	1	11	3	1
1:15	1:21	6	To adjust distance	4	..	..	..	..
1:25	1:58	33	To tighten tools	7	2	4	3	40
2:05	2:09	4	To tighten tools	2	..	..	..	..
2:11	2:20	9	To tighten tools	1	..	..	..	..
2:21	2:25	4	End of run.....	..	2	6	3	57

Total time of test, 8 hr. 31 min. Average progress, 7.6 in. per hr. for running time of 3 hr. 57 min. Average progress for 8 hr., 3¾ in. per hr.

River Diversion and Flood Control in Missouri

SYNOPSIS—This $4,000,000 job is to reclaim nearly 1,000 sq.mi. of low land. Flood waters now spreading over this district will be diverted to a defined channel, with large detention basins to equalize the flow. The use of electrically operated excavators of great size is a feature of the work. The plan and methods are described fully.

The reclamation of an area of some 500,000 acres of swamp and overflowed land in southwestern Missouri—constituting the Little River drainage district—is being effected by two distinct classes of work: River diversion to prevent the entrance of flood water, and ditches to drain the swamps and take care of the rainfall. The project was described in *Engineering News,* Mar. 18, 1915, and work is now in active progress. The general situation is shown by the plan and profile, Fig. 1.

The district lies back from the Mississippi River; it has a length of about 90 mi., north and south, and an average width of about 10 mi. The floods come from the Castor River and the Little River (or Whitewater), which flow out of the Ozark hills near the northwestern corner of the district. These streams are of a flashy character, but after leaving the hills the water flows slowly southward through the district by tortuous and ill-defined channels. These streams will be intercepted by a diversion channel along the north boundary of the district and extending eastward to the Mississippi River.

The surface waters will be carried southward by the ditch system, which will extend to the Arkansas line and there discharge into Big Lake, Arkansas. The state boundary is an arbitrary limit to the Little River district. On the Arkansas side, however, the Keystone drainage district has been organized, which is practically an extension of the other. Its works will deliver the combined waters from the two districts into the St. Francis River at Marked Tree, Ark.; this in turn discharges into the Mississippi above Helena, Ark.

The Little River drainage district has its headquarters at Cape Girardeau, Mo. William A. O'Brien is Chief Engineer; N. C. Frissell, Assistant Chief Engineer; B. F. Burns, Office Engineer. The river-diversion work is divided between two resident engineers—L. D. Sheppard and Harry V. Newcomb. The drainage-ditch system is divided among three resident engineers—Blair Boyle, Howard Henderson and E. S. Blaine, with William Mulholland as Locating Engineer. Isham Randolph (Chicago) is consulting engineer for the district. John A. Ockerson (St. Louis), A. E. Morgan (Dayton, Ohio) and Daniel W. Mead (Madison, Wis.) are consulting engineers for the bondholders.

The contract for the whole of the river-diversion work has been let to the Floesch Construction Co., of Rochester, N. Y., at $1,260,000. S. T. Neely is General Manager for the company. The drainage-ditch work is divided among several contractors at an aggregate price of $2,706,000. The total amount of excavation will be

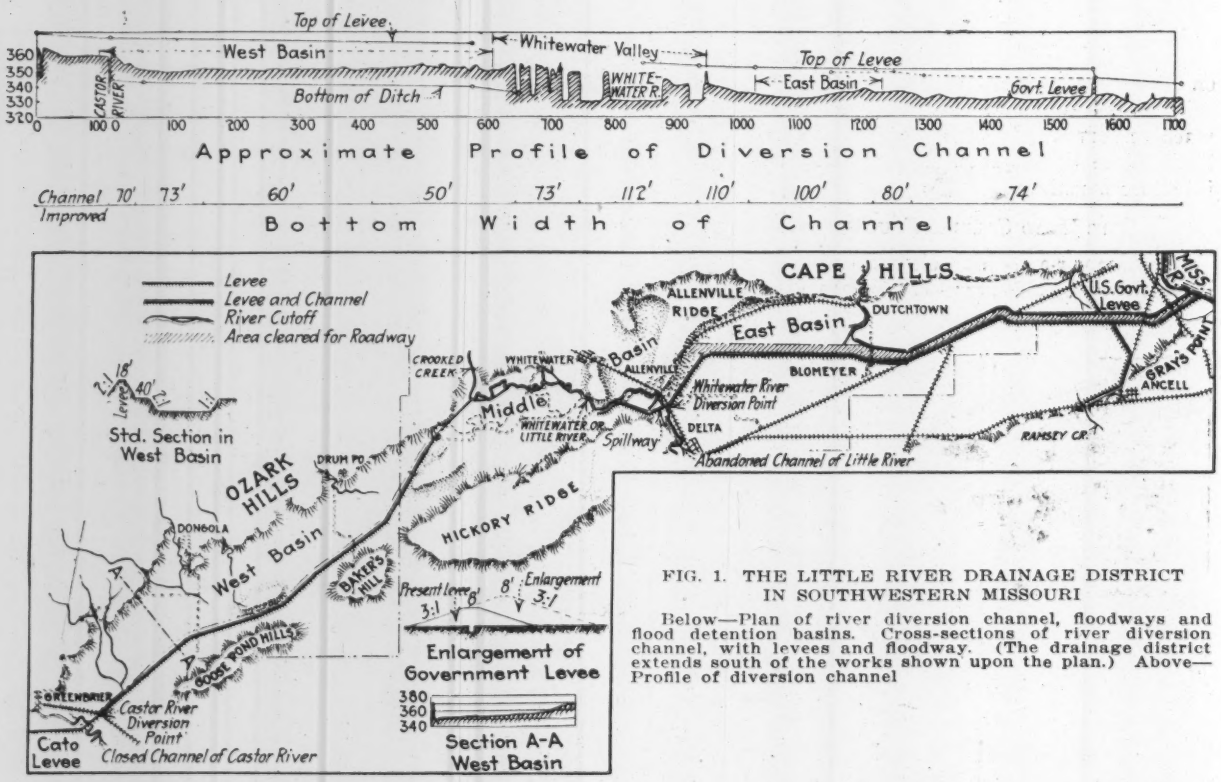


FIG. 1. THE LITTLE RIVER DRAINAGE DISTRICT IN SOUTHWESTERN MISSOURI

Below—Plan of river diversion channel, floodways and flood detention basins. Cross-sections of river diversion channel, with levees and floodway. (The drainage district extends south of the works shown upon the plan.) Above—Profile of diversion channel

8,600,000 cu.yd. for the river diversion and 34,230,000 cu.yd. for the drainage ditches. The former work is expected to be completed by October, 1917, and the latter by January, 1918.

River Diversion for Flood Control

The method of eliminating the flood water from the district is to carry the two rivers eastward across the north end of the district in a channel extending to the Mississippi River, as previously mentioned. Beginning near Greenbrier (Fig. 1), the channel of the Castor River will be improved for 2.5 mi. to the point of diversion, beyond which a channel and levee will extend 12.3 mi. to Crooked Creek. High ground here makes a levee unnecessary, but the channel of this river and then of the Little (or Whitewater) River will be improved and straightened for 5.5 mi. to near Allenville. There the channel and levee work will recommence, extending thence to the river, 15 mi. (as shown by the plan in Fig. 1). The total length of the diversion channel is 32.5 mi.

Typical sections are shown in Fig. 1. The main-channel section has a bottom width of 50 to 112 ft. (as shown beneath the profile in Fig. 1), with slopes of 1:1 on the open side and 2:1 on the side adjacent to the levee. There is a 40-ft. berm on this latter side. The levee is 8 ft. wide, with an 8-ft. crown and side slopes of 3:1. It has a central muck ditch 8 ft. deep and 6 ft. wide, which insures thorough bonding with the ground beneath. For the floodway extending from the East Basin to the river the smaller levee, along the north side, has an 8-ft. top, with side slopes of 2:1 and a muck ditch of the size already given.

To provide for the possibility of a flood in the upper basin of the diversion channel being beyond the capacity of the channel, it is proposed to establish a spillway in the levee near Allenville. This would discharge excess water into the abandoned channel of the Little (or Whitewater) River, and the amount would be much less than the bank-full capacity of this channel. The spillway would be a stretch of levee with its crest 5 ft. below the normal crest elevation and having the slopes paved with concrete. Concrete toe walls, perpendicular to the slope, would prevent undercutting or wash.

Floodways and Detention Basins

The provision for heavy floods is an interesting feature of the work. The channels could not be made large enough to carry the water of maximum floods, but a strip of land has been obtained along the channel as a floodway. This is cleared and grubbed and will be flooded when the channel capacity is exceeded. On the west end of the work this floodway has a width of about 1,500 ft. and is bounded by rising ground (Fig. 1).

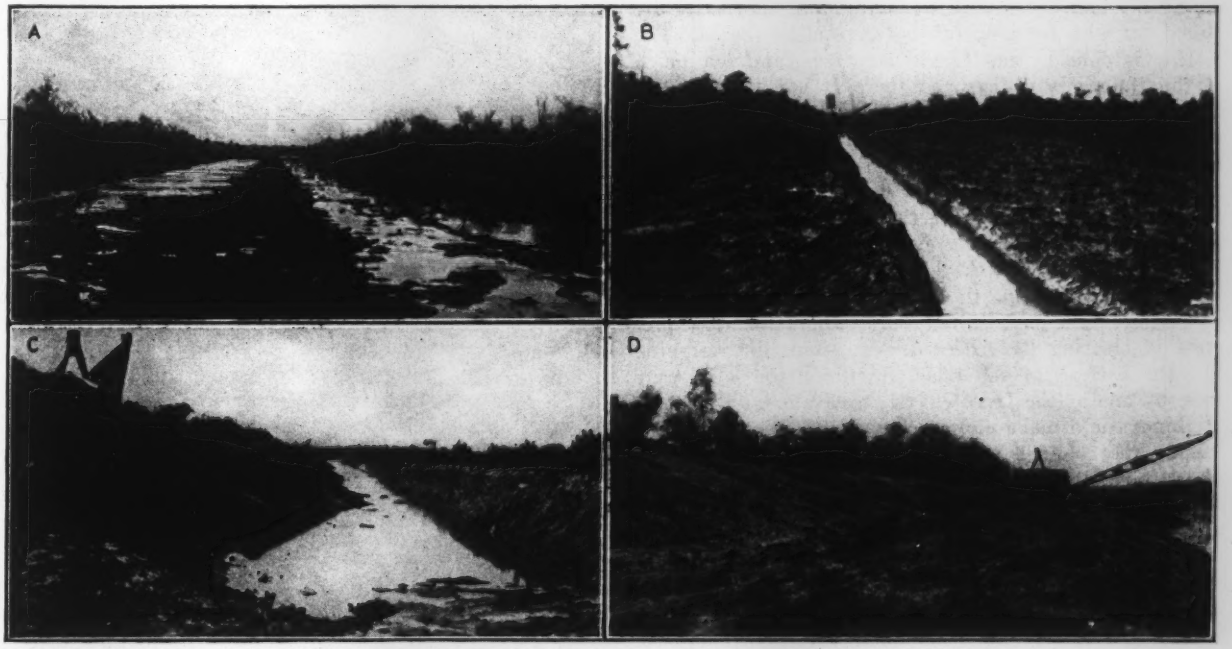


FIG. 2. DIVERSION OF THE LITTLE RIVER, IN MISSOURI

A—Area stripped for the channel and levee. B—Muck ditch for levee. C—Cutting the channel; the excavator has commenced the cut, another (not shown) is completing it to full section. D—Levee and channel; the excavator is on the 40-ft. berm, excavating the channel at the right and putting the material in the levee at the left

At the east end of the work, the ground is low, and the floodway has to be formed by a second and smaller levee on the far side of the cleared area. As this excludes flood water from the adjacent district on the north, a drainage ditch outside of and parallel with this levee is required to care for surface water for the land north of the floodway. The width of this defined floodway is about 900 ft.

Besides the floodway there will be three flood-detention basins, formed by the levees, having an area of 23 sq.mi. and providing ample capacity to hold the water of a sudden storm on the watershed. These basins will be dry and in large part available for cultivation except during flood periods.

The use of these large basins is a special feature of the headwater-diversion system, and it is believed that the formation of detention basins of such magnitude has not been attempted elsewhere in this country or abroad

in connection with land-reclamation works. The capacity of the West Basin is about 3,000,000,000 cu.ft. and 5,000,000,000 cu.ft. at elevations 360 and 365 (see profile, Fig. 1); that of the Middle Basin is nearly 2,000,000,000 cu.ft. at the former elevation. The capacity of the East Basin is 1,500,000,000 cu.ft., of which 418,000,000 cu.ft. is above the Mississippi River flood plane of 1,844, so that this basin provides storage capacity for floods of the hill streams even when the Mississippi is in flood.

Levee and Channel Construction

The site for the levee and channel is cleared of all vegetation. Trees are felled by means of crosscut saws and cut up into 20-ft. lengths. The stumps are either pulled out bodily by a Clyde steam stump-pulling machine, with occasional blasting of large stumps, or they are all blasted, using 40% dynamite, and the pieces are pulled out with horse-operated stump-puller tackle. The cutting is done by small local contracts, and not much of the timber is utilized, as there is not time for its sale and removal in advance of the excavation. The logs and slash are either stacked on the channel site and burned or are stacked outside of the levee line and covered with the stripping. The steam stump puller does its own stacking, but where the material is handled by teams the stacking is done by steam hoists or skidders.

The entire area for the levee and channel is stripped to a depth of 12 in. in advance of the main-channel excavation. On part of the work, where the slow progress of clearing had checked excavation, the stripping and the cutting of the muck ditch for the levee were done by one of the large electric excavators. But these machines could make more money for their owners when cutting the channel, and the best results were obtained where the clearing was kept well in advance, and a special stripping excavator was used between this and the main excavators.

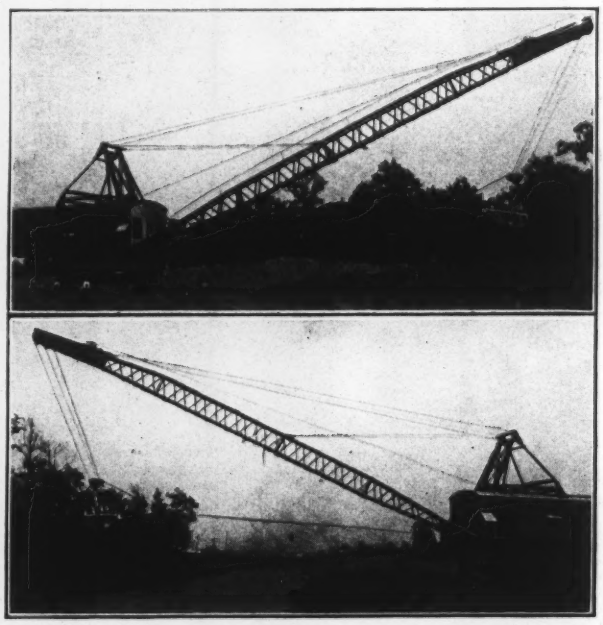
The stripping machines are 80-ton Bucyrus dragline excavators with 70-ft. boom and 1½-yd. bucket. Each is mounted on two pairs of caterpillars. Even the broad base of the caterpillars is not sufficient to support the machines in traveling over the soft ground, full of stump holes. They travel therefore on pads made of layers of heavy planking, the pads being 16x5 ft. in size and handled by the boom. This machine works back and forth, longitudinally of the work, stripping the entire surface, cutting the muck ditch and removing any broken stumps left by the stump puller.

Where all the work is done by the main dragline excavators, the method of procedure is as follows: One of the excavators travels first along the site of the south berm, stripping the surface of the channel and depositing the material in a spoil bank 40 ft. from the north edge of the channel. This bank is about 4 ft. high, well rounded, and drainage openings will be cut through it later to admit surface drainage from the floodway area to the channel.

After stripping the surface for a distance of about 2,000 ft. the machine returns along the center line of the levee, stripping the levee site and making the muck ditch (8 ft. deep and 6 ft. wide). The stripping is thrown upon the stumps and grubbing piled 15 ft. from the south toe of the levee. The material from the muck ditch is placed along the south toe, so as to retain the main fill.

Having finished this work on the stretch of 2,000 ft., the machine then moves to its first position (on the berm) and again travels forward, excavating one-third of the channel prism. The second excavator follows and completes the channel to its full dimensions (Fig. 3). Both machines build the levee with the excavated material. The levee has a maximum height of about 18 ft. and is built in three layers or lifts of 5 to 6 ft.

For the work on the lower part of the river diversion, where the stripping is done by a separate machine, the method of working is slightly different. The stripping from the channel is placed outside of the levee, along the edge of the right-of-way. The stripping from the levee site and the material from the muck ditch are placed along the toe of the levee on the berm or channel side. The main dragline machines then travel along the berm,



FIGS. 3 AND 4. ELECTRIC DRAGLINE EXCAVATORS WITH 100-FT. BOOMS AND 4½-YD. BUCKETS

Fig. 3—Dumping on the levee. Fig. 4—Finishing the river-diversion channel

excavating the channel and building the levee in four lifts. Fig. 2 shows some of the stages of the work.

On the upper part of the work, above Allenville, the material is mainly a stiff white clay, which stands very well in the slopes and is easily dressed to the specified slopes both for the channel and the levees. There are some red earth and sandy soil also, while at some points, generally in high ground, the bottom of the cut is in fine sand. The dragline machines, with diagonal cuts, finish the channel very closely to the specified slopes. For finishing the levee the lighter material is used in the top and sides, any large lumps being broken up and leveled by a man with a shovel. The slopes will be seeded with a mixture of white clover, bermuda and herd grass.

The lower section of the work is largely in gumbo soil, which will stand in the slopes of the cut, but when heavily saturated will not stand at the specified slopes for the levee. Here the material is being deposited temporarily as a spoil bank, of lower height and greater width than the levee section. After the material has dried and settled, a dragline excavator will travel along the

top of this bank, taking the material from the sides and placing it on the top, giving the required height and slopes.

For the drainage ditch outside the north levee forming the floodway the stripping is done by a steam caterpillar dragline with 1½-yd. bucket. This places the material along the south toe of the levee. A steam dragline excavator with 100-ft. boom and 3½-yd. dipper then excavates the ditch and throws the material into the levee. This has a slope of 2:1 on the ditch side and 2¾:1 on the floodway side; the flatter slope is due to the material spreading out over the stripping piled along the toe.

Electric Dragline Excavators

The main-channel excavation is done by four Bucyrus electrically operated dragline machines, two on each division. This use of electric power for large excavators is one of the specially interesting features of the work. Two are 165-ton machines, having 100-ft. booms and 3½-yd. buckets, with 200-hp. main motor and 115-hp. swinging motor. Of the remaining two, one has a 100-ft. boom and 4½-yd. bucket, while the other has a 125-ft. boom and 3½-yd. bucket. These latter have 250- and 125-hp. motors for the main operation and swinging movements and weigh 206 tons each. The machines are shown in Fig. 3.

All these machines operate on 3-phase 60-cycle 440-volt alternating current, which is generated at 13,200 volts at the power house of the Cape Girardeau (Mo.) electric light plant, additional turbo-generators being installed for the purpose. The contractor builds his own transmission line along the work, using steel latticed poles. At each excavator there is a transformer outfit. For the machines on one section the outfits are mounted on skids and hauled ahead by a horse-operated windlass and cable as the work progresses. At first the excavator, in moving ahead, picked up its transformer outfit and swung it ahead, but this involved too much loss of time in the work of excavating. On the other section the transformer outfits are on scows floating in the first cut made by the excavators.

Each electric excavator has an operator and an oiler. The former sits at the front and controls all movements by a bank of levers. The latter sees that the machinery is kept in working condition. These men work three 8-hr. shifts. There is no stop for meals, but the oiler (glad to learn how to run the machine) takes a spell while the operator eats his lunch. There are four to six pitmen to each machine; and they work 12-hr. shifts.

An electric fan near the operator's seat serves to keep the mosquitoes and flies from troubling him. Work proceeds continuously, night and day, for six days a week, and on Sundays the machines are overhauled and given any necessary repairs.

The main part of the machine revolves on a 24-ft. turntable upon a steel tower about 24 ft. square and 10 ft. high. The total weight is carried by four 4-wheel trucks with 22-in. grooved wheels. The two forward trucks are pin-connected to the tower. The rear trucks are connected by a long transverse equalizing beam, so that the machine is stable even on irregular track. The machine makes a full swing (loading, dumping and return) in 45 to 60 sec.

The trucks ride on two tracks of 30-in. gage, spaced 16 ft. c. to c. Each track is in lengths of 15 ft., which are shifted by the machine. The tracks are of very heavy construction, but soon become distorted vertically, owing to the weight of the machine and the soft soil. Each track has 100-lb. rails on steel base plates on longitudinal timbers 10x12 in. These timbers rest on closely spaced 6x12-in. ties, 8 ft. long, bolted to 4x12-in. longitudinal planks. Ring bolts in the platforms or pads are provided for the attachment of chains in shifting. (*Engineering News*, July 13, 1916.)

The maximum monthly records for these electric dragline excavators are 90,000 and 80,000 cu.yd. for machines with 4- and 3½-yd. buckets respectively. For a 24-hr. run the maximum record is 5,800 yd.

Government Levee

The work crosses the present Government levee, which will be cut for the channel and the remaining portions widened and raised to the full section of the new levees (Fig. 1). This involves also the construction of a diversion channel for Ramsay Creek, outside the Government levee, and with a new levee along its outer side.

This work is being done under subcontract by Bartson & Millard, of Peoria, Ill., using a Monighan steam dragline excavator with 100-ft. boom and 3½-yd. bucket. For the stripping and muck-ditch excavation they use a Lidgerwood dragline machine; this travels on rollers and has a 60-ft. timber boom and a 1-yd. bucket.

The railway bridges across the diversion channels and ditches are built by the railway companies, the structures being needed to meet the requirements of the engineers for the district. The state law compels county authorities to pay for bridges across drainage channels, but one of the counties concerned has claimed that this does not apply to the river-diversion work. This question is now before the Supreme Court of Missouri, and in the meantime the district has built trestles to carry roads across the channel.

Several railway lines traverse the district, so that in most cases the excavators could be assembled at a point of shipment and then be moved a short distance to the work.

Engineering and Inspection

The field party on each of the two river-diversion divisions consists of an instrumentman, rodman, chainman and inspectors. The first three stake out the work in advance and make cross-section measurements of the finished work. Four lines of stakes are set; these mark the center lines of channel and levee and the toes of the levee slopes. Any labor necessary to assist the party is furnished by the contractor.

Cross-section measurements are plotted directly in the field on section-paper tablets, with carbon paper for making a duplicate. Thus any mistake or discrepancy is evident at once. Three shots are taken on each slope and at 5-ft. intervals across the bottom. On one stretch of very wide ditch the bottom was too wet for wading, and there was not enough water for a boat. Here two 100-ft. tapes were tied together and stretched across the cut, while a level rod was suspended in such a way that it could be worked along the tape. The measurements were taken from the water level, as the grade line, so that the sag of the tapes had no effect upon the readings.

Monthly records are made of the expenses on each residency, showing the ratio of engineering expenses to amount of excavation.

Loose-Leaf Filing System for City Surveys

By James M. Owens*

After the great conflagration of 1906, which destroyed practically all of the municipal records in San Francisco, it was found when the scattered forces of the city engineer's office were brought together in a temporary office (in a hay and grain barn) that there remained hardly any official records with which to recommence business. During the period of reconstruction there was not much time available for refinements in the handling of office work. Particularly was this true in the case of the Survey Department. It was quantity rather than quality that the public demanded, and there was neither time nor money available for introducing modern office equipment or methods. During this time the need of a new system in handling this

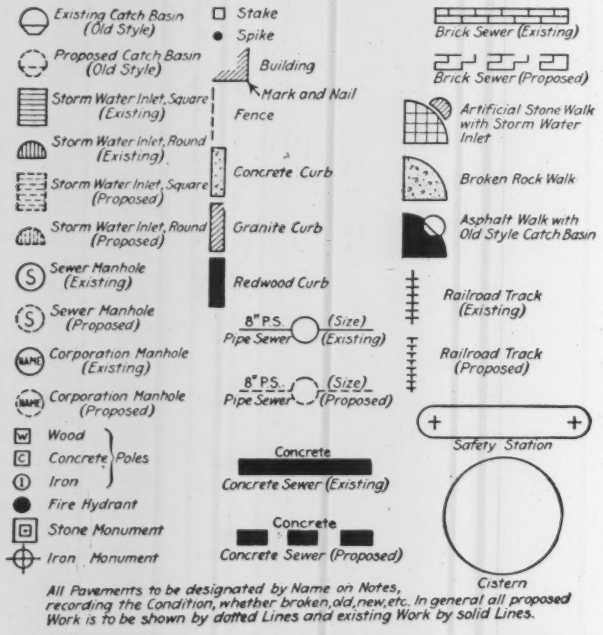


FIG. 1. CONVENTIONAL SIGNS FOR DIFFERENT CLASSES OF STREET WORK

office work was very apparent. A loose-leaf system for field notes promised satisfactory results. As an index for these a 15x20-in. lithographed copy of a map compiled by the city assessor proved convenient.

Briefly, the assessor's idea was to divide the city into some 18 districts, starting with the older 50-vara district. A certain allotment of numbers was allowed for each district. In the partly developed districts an extra set of numbers was made available to cover all possible block extensions. The block numbers on this map were from 1 to 7,200. Under this system each block has a definite number and location, which remains unchanged despite the block extensions that may be made. This gives a very simple method of locating a particular property when the block number is once known. In adapting this system to the needs of our surveying department the following simple scheme of indexing was used:

For single units of street work—say a roadway pavement or crossing—the highest and lowest block number adjoining the work was used; such as 24/48, the survey being filed under 24. For more extended pieces of construction work, covering a series of blocks, the lowest block number adjoining the work and the highest within its scope were used in the same fashion. For property surveys the block number was used. For special projects

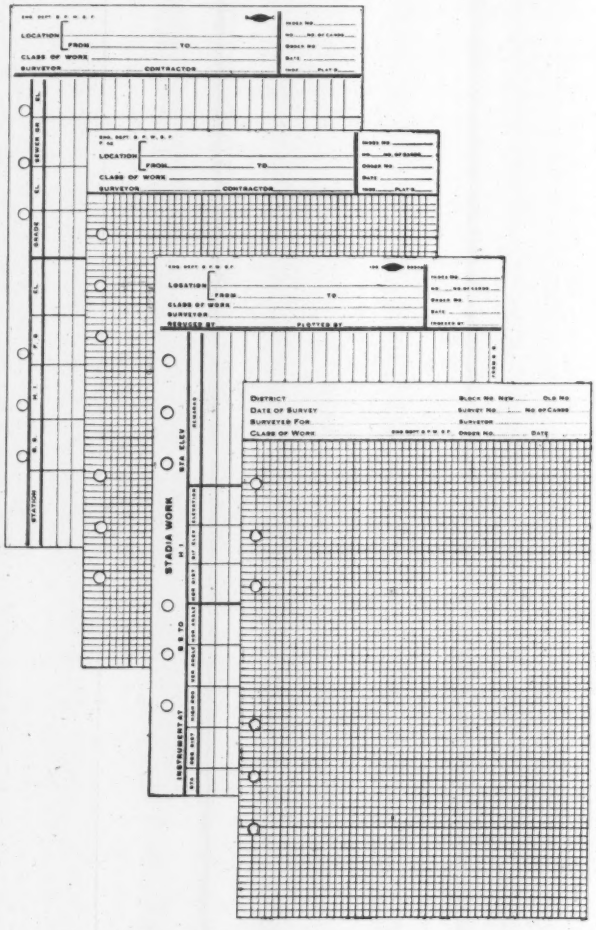


FIG. 2. FOUR LOOSE LEAVES (5x7¾ IN.) FOR RECORDING SURVEYS

such as the various tunnels and municipal railroads, separate index cards were made, filed separately and designated by the name of the particular project. This briefly describes our system of filing field notes. This method has been in use here for the last 2 yr., and we have found that it works very satisfactorily.

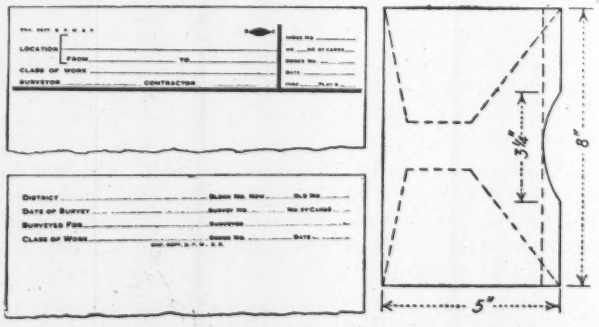


FIG. 3. ENVELOPES FOR STREET (UPPER) AND LOT (LOWER) SURVEYS

*Assistant City Engineer, in charge of pavement design, San Francisco, Calif.

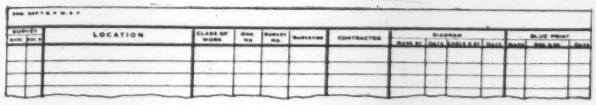


FIG. 4. SURVEY RECORD IS KEPT ON THIS 5x8-IN. CARD

For the sake of comparison it may be well at this time to give an illustration of the working of the old system and the disadvantages incident to its use. Whenever a survey was ordered, it was found necessary to give the surveyor from one to six notebooks as references for his daily work, that he might correlate it with surveys already completed. When this daily assignment of work in the field was completed he turned in his notes all in one book. This work was then assigned to the different draftsmen for the preparation of diagrams. Under this order of things the surveyors would have in the field for an entire day, say from 20 to 40 fieldbooks, containing some 300 or 600 jobs, notwithstanding the fact that for their immediate needs only one job in each book might be required. For the office men who were making up the notes almost as many books were in daily use.

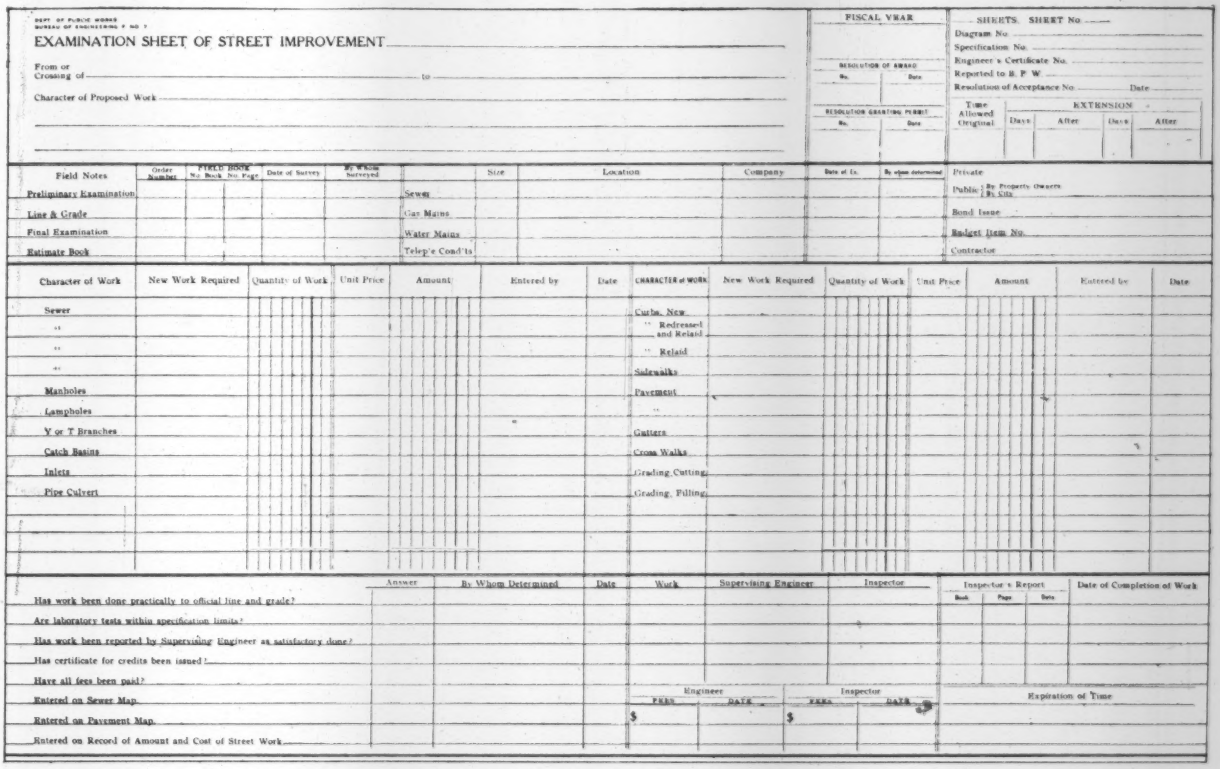
The transition from the old to the new system presented some slight difficulty to the field men for the reason that it is always more or less difficult to get away from the beaten path. They soon learned, however, to appreciate the greater simplicity of the new system.

No claim is made to originality in the use of a loose-leaf system for this class of work. Claim is made only for the special arrangement of the handling and filing of the notes. The following description will give a good idea of the arrangement of our filing system for field surveys, the component parts of which are as follows:

Order books, consisting of a General Order Book and a Municipal Order Book.

A 20x30-in. cardboard assessor's desk map of the city, with plate-glass cover.

Specially designated loose-leaf covers, with stiff backs,



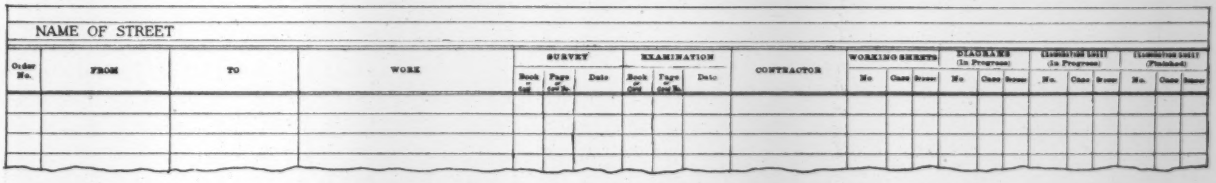


FIG. 5. (UPPER) EXAMINATION SHEET (9½x15 IN.) GIVING FULL DATA ON STREET IMPROVEMENTS. (LOWER) GENERAL INDEX CARD (9½x11 IN.) OF STREET WORK

consecutively numbered and properly labeled.

Set of field instructions for surveyors.

Sets of conventional signs for the different classes of street work (Fig. 1).

Four different types of loose leaves for recording surveys (Fig. 2), 5x8-in.

Two different types of filing envelopes for surveys (Fig. 3).

Special card for arranging data from the inception to the delivery of diagrams (Fig. 4), 5x8-in.

Assignment-of-work folder (same as top sheet in Fig. 2).

Work-to-be-checked folder (same as top sheet in Fig. 2).

Work-to-be-filed folder (same as top sheet in Fig. 2).

Special celluloid index cards tabbed and numbered in series of 100.

Supplementary celluloid index cards tabbed and numbered in series of 20.

Two special sizes of tracing-cloth sheets for diagrams 9½x15-in. and 9½x42-in. (Fig. 6).

Extra sheet for street work (Fig. 5, upper).
General index card street work (Fig. 5, lower).
Standard 5-drawer built-up sections for filing surveys.
Case for filing field diagrams and blueprints of finished work.

Orders for street and lot surveys are of two classes—"General" and "Municipal." These orders are taken from the desk books by the assistant in charge of the prepara-

[A] NOTICE TO CONTRACTORS AND INSPECTORS
All street monuments found along the line of the work must be capped with iron frames.
Frames will be furnished by Engineering Department upon application.
Street monuments shall not be disturbed until permission is given by this office.

[B] CITY AND COUNTY OF SAN FRANCISCO, DEPARTMENT OF PUBLIC WORKS, BUREAU OF ENGINEERING
M. M. O'Shaughnessy, City Engineer
Surveyed for.....
Date .....
Drawn by.....Checked by.....
Date.....Date.....

[C] No. .....
Order No. .....
Survey No. .....
No. of cards.....
Surveyor .....
Private Contract .....
Public Contract .....

FIG. 6. CAPTIONS WHICH ARE ATTACHED TO TRACINGS FOR STREET-WORK DIAGRAMS

tion of field data, who prepares the necessary information for the surveyors. These are turned over to the assistant in charge of filing for distribution to the different surveyors. These the latter place in their folders, together with sufficient blanks for the work in hand. Upon completing a survey the reference notes and finished notes are turned in and are properly indexed and filed by the assistant in charge of filing.

In arranging the field notes in the cabinet the following method is used: Block numbers are arranged consecutively. For surveys of property in the same block, with the block number as prime unit, the order of filing is according to the chronological sequence of the surveys (their order number). An index of these surveys is made in the Daily Record Sheet, and given to the assistant in charge of street work, who assigns the different pieces of work to the draftsmen. When any particular job is completed, the draftsman turns over all working data, notes and completed diagrams to this assistant, who places the notes in the "To-Be-Checked" folder, and has the job checked as soon as possible.

When checked the notes are placed in the "To-Be-Filed" folder, and at the end of each day are filed away in the cabinet in their respective envelopes. The completed diagram is then blueprinted; three prints being made, one for the Supervising Engineer, one for the inspector and one for the contractor. The filing envelope, after being made and placed in the cabinet, always remains there and acts as a receipt and check for the number of surveys made.

This is the system for all surveys made; in special cases, where the notes are extended and are required in the field by the construction engineer and his inspectors, photostat copies are made of each survey. When a large job is completed, a set of these notes is filed in a folder, with the rest of the working plans and data, and consequently a complete history of any particular job is always available.

On my recommendation this system was adopted for the division of surveys, the files of which are kept distinct from those of the construction, sewerage-design, water-supply, municipal-railway, tunnels, and other divisions of the Bureau of Engineering, each of which has its separate system. All divisions are under the direction of M. M. O'Shaughnessy, City Engineer.

Some Results of Analyses of Garbage and Waste

The thorough investigation and study of the collection and disposal of municipal waste in the District of Columbia, made by Irwin S. Osborn, Consulting Engineer, Cleveland, Ohio, makes available data on the composition and value of these wastes which will prove useful to others studying the same problems elsewhere. An outline of the methods employed in this investigation was given in *Engineering News*, Aug. 17, 1916. Some of the results of the investigation follow.

Garbage Statistics

The average per capita per annum production of garbage in the District at the present time amounts to approximately 285 lb. From the studies made it was found that the weight per cubic foot of garbage varies considerably from day to day, depending upon how well the load is packed. Where collectors deliver two or more loads per day, it is found almost invariably that the first load runs heavier per cubic foot than the second load, which is simply placed in the wagon loosely, although in cold weather the unit waste is about the same. The daily average weights per cubic foot run from 44.7 to 51.2 lb.

The per capita production of garbage in the District of Columbia is very high as compared with the per capita productions of other cities. This condition can be accounted for by the fact that there are practically no slums in the city and the majority of people are of the better residential class, who produce a larger percentage of waste than do residents in average cities. At the same time all the garbage is collected, including that from hotels and restaurants, as well as from the floating population, whereas in the majority of cities the garbage from hotels, restaurants, etc., is gathered by private collectors and is not included in the reports. The per capita production varies from year to year, although increasing each year with few exceptions from 1900 to 1915.

Production of Rubbish

The production of rubbish has increased considerably during the past years, as shown by the records. In 1907 approximately 0.196 cu.yd. was collected per capita per year, and in 1914 the corresponding amount was 0.415 cu.yd. The latter figure, no doubt, is as close to the maximum as can be expected.

In making weight and volume studies it was found that there was considerable variation in weight per cubic yard, the largest occurring during rainy weather. The variation per cubic yard was due either to moisture in composition of the material or the packing of the load by the collector. From the total studies made, the average weight per cubic yard was found to be 201 lb.

The results of analyses (see accompanying Table 2) to determine calorific values, by the United States Bureau of Mines, constitute the first data which are available in regard to the calorific values of various constituents of rubbish. From the analyses made it was found that the amount of ash in cardboard, linoleum and magazines was considerably higher than that found in other items. It was the opinion of the fuel chemist of the Bureau of Mines that the amount of ash in cardboard and linoleum was no doubt due to mineral fillers, and a high ash value

TABLE 1. RESULTS OF MECHANICAL ANALYSES OF RUBBISH IN % OF WEIGHT, 1914-15

Material	Nov.	Dec.	Jan.	Mar.	May	June	July	Aug.
Newspaper	10.4	17.8	19.5	16.1	15.6	17.2	17.0	16.0
Manila paper	6.6	12.7	7.0	9.5	8.7	7.3	12.1	13.4
Cardboard	0.8	9.4	11.7	10.5	9.7	6.9	10.9	12.8
Books, etc.	4.4	3.0	2.4	2.4	3.3	3.2	1.4	5.6
Mixed paper	16.6	4.4	3.1	5.0	4.6	2.8	2.4	2.6
Rags	5.9	3.7	4.5	5.2	7.2	4.3	5.0	5.7
Wood	1.2	7.4	1.4	2.8	2.6	4.1	3.6	4.1
Leather	1.5	0.2	0.4	0.7	1.2	0.2	1.0	1.1
Rubber	0.3	0.3	...	0.3	0.5	...	...	...
Screenings	...	4.6	18.5	13.4	10.4	18.6	12.3	11.1
Tinware	10.2	7.3	11.7	13.6	8.8	6.6	5.6	6.4
Enamelware	0.4	0.4	0.1	0.4	0.3	5.0	...	0.1
Metals	1.8	0.6	0.3	0.8	1.3	1.1	0.8	1.5
Bottles	11.0	8.1	7.7	9.0	7.7	8.3	8.4	7.2
Broken glass	3.8	3.5	4.0	4.7	5.2	4.3	4.8	3.5
Excelsior	...	0.7	0.3	0.8	0.7	0.1	0.7	...
Mattresses, etc.	5.2	1.1	...	...	0.3	...	0.7	0.1
Matting	3.1	0.8	0.1	0.3	1.6	1.5	1.4	...
Linoleum	0.7	...	...	...	0.3	0.1	...	0.1
Straw	...	...	0.4	...	...	...	...	...
Dirt	16.1	14.0	6.9	4.5	10.0	8.4	11.5	8.7
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

in the rubber resulted from the clay and similar filler, as well as the fabric contained in the rubber. The high ash value of books and magazines was attributed to the sizing of the paper, which would account for its low calorific value.

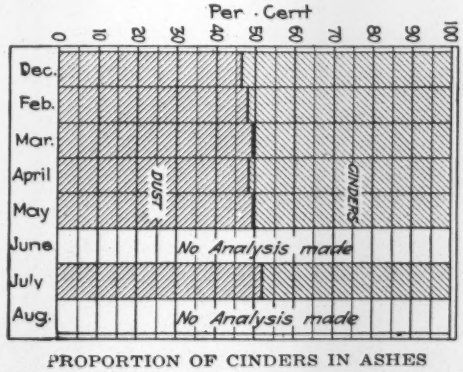
From the results obtained in making calorific tests of the various component parts of the rubbish it is possible to compute the calorific value of the rubbish as collected. The percentages of the various constituents determined by making mechanical analyses, multiplied by the calorific values obtained, give the average calorific value of the rubbish as collected.

In comparing the calorific values with the average constituents found in the rubbish as a whole, taking into account all items determined by mechanical analyses, it was found by computation that the average calorific value in samples as collected, including the noncombustibles, such as metal, dirt, etc., amounted to 5,500 B.t.u. per lb.

Analyses of Ashes

In making the analyses of the ashes it was found that there was no definite relation between the weight of the cinders and the weight per cubic yard of the ashes, although the weights show a slight range per cubic yard. In cases where the maximum weights were found, the ash contained a higher percentage of moisture, the percentage of cinders depending on the residences from which it was collected, as would be expected.

The percentage of cinders produced in the ash was approximately 50%. On comparing the results given in



PROPORTION OF CINDERS IN ASHES

tables of calorific value it was evident that considerable combustible material was left in the ash. The average of 45 analyses of the screened-out cinders showed the following: Moisture, 2.05%; volatile matter, 3.53; fixed carbon, 51.1; ash, 45.3%. The average B.t.u. per lb. in the samples as received was 7,858; and dried, 7,932. The

TABLE 2. RESULTS OF CALORIFIC TESTS ON COMPONENTS OF RUBBISH, BY BUREAU OF MINES*

Component	Moisture	Dry Coal: Volatile Matter	Dry Coal: Fixed Carbon	Dry Coal: Ash	Dry Coal: Sulphur	B.t.u. as Received	B.t.u. Dry Coal
Newspaper	6.2	86.9	10.0	3.1	0.10		8,230
	8.67	87.41	7.84	4.75	0.11	7,485	8,196
Manila paper	5.1	88.9	8.6	2.5	0.25		7,840
	8.99	89.12	8.14	2.73	0.13	7,626	8,379
Mixed paper	5.3	84.8	10.7	4.5	0.15		7,910
	9.07	86.85	9.6	3.55	0.16	6,867	7,552
Books and magazines	4.4	71.2	5.1	23.7	0.35		5,630
	7.24	74.72	4.88	20.4	0.08	5,400	5,821
Cardboard	6.0	80.3	10.4	9.3	0.15		7,430
	8.67	84.39	9.33	6.28	0.16	7,154	7,833
Screenings	8.6	78.1	11.4	10.5	0.30		6,910
	9.97	83.11	9.08	7.81	0.46	6,483	7,201
Wood	6.3	87.6	11.0	1.3	0.10		8,910
	9.55	87.78	10.22	2.0	0.19	7,721	8,536
Excelsior	7.0	88.5	10.4	1.1	0.05		8,580
	9.48	92.76	6.8	0.44	0.19	7,853	8,675
Matting	10.20	82.41	8.08	9.51	0.19	6,266	6,978
Rags	3.7	89.9	8.8	1.3	0.10		7,410
	6.47	94.39	2.45	3.16	0.16	7,312	7,818
Rubber	1.2	...	...	47.6	0.65		6,620
Leather	5.9	72.3	17.7	10.0	0.45		8,530
	10.33	78.64	11.18	10.18	0.45	8,240	9,189
Dirt	2.7	20.2	9.1	70.7	...		4,261
	3.7	22.27	3.87	73.86	...	3,661	3,802
Linoleum	2.1	65.8	6.8	27.4	0.40		8,310
Mattress	6.4	75.4	18.5	6.1	0.25		7,430

*The average calorific value per pound of rubbish as collected, including dirt, metals and other noncombustible material, amounts to 5,500 B.t.u.

calorific value of this ash, if recovered each year, would amount to that produced by approximately 24,000 tons of coal. The highest calorific value of ashes was found in the collections from the residential sections, which contained a higher percentage of unburned coal.

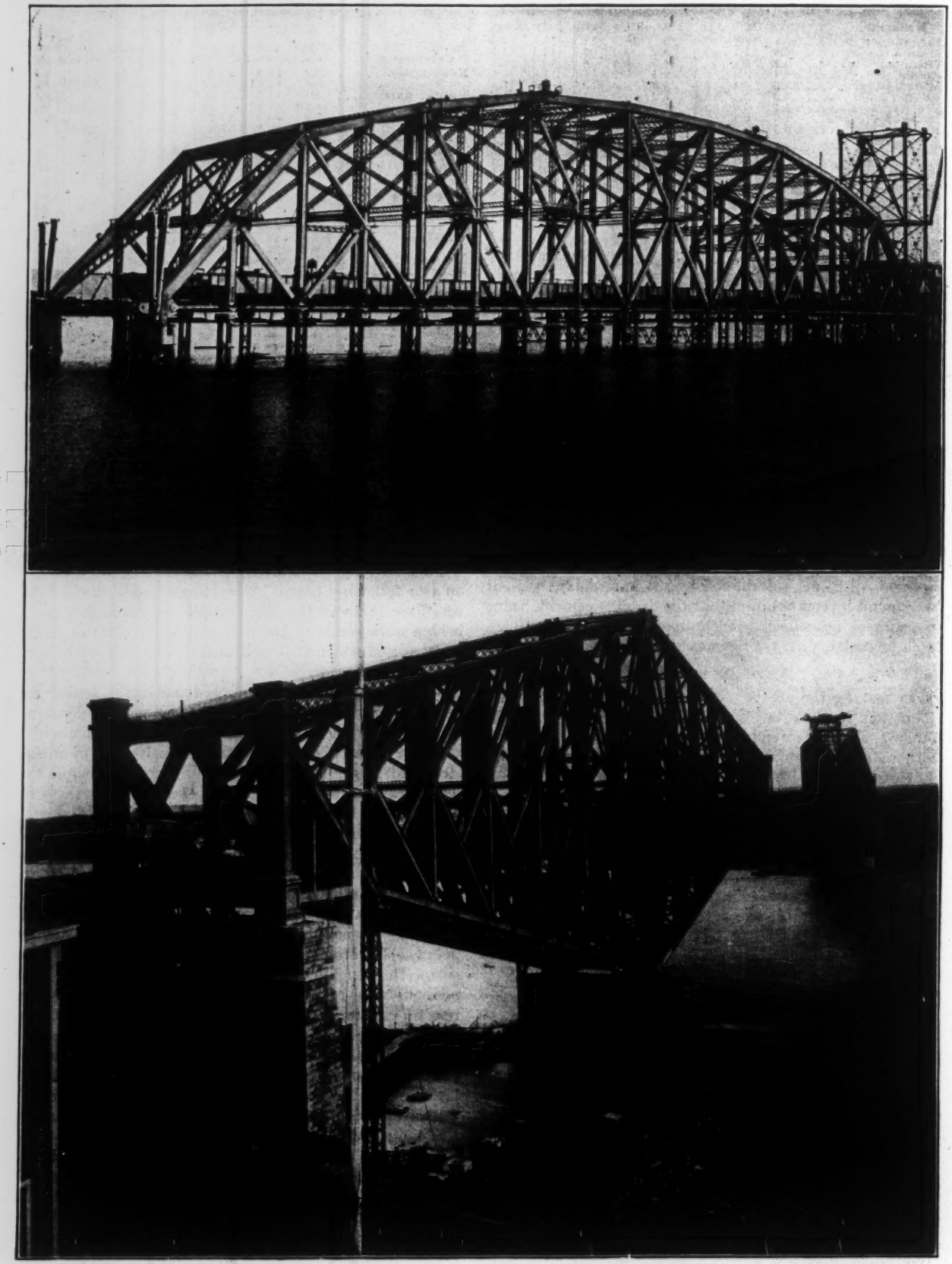
Quebec Bridge in Picture

Four striking views of the Quebec Bridge are presented on two following pages. They immediately recall to all who have followed this, the world's most notable piece of bridge work, some of the outstanding features of design and erection which it is well to review as the day of completion draws near.

One view shows the famous K-truss in the anchor and cantilever arms, the first use of an epoch-making departure in design. With this goes the novelty of a bridge within a bridge to carry the floor, the extensive use of nickel-steel and larger and longer rivets than ever before. A second view shows the 640-ft. suspended span, in itself the longest single-span bridge yet completed.

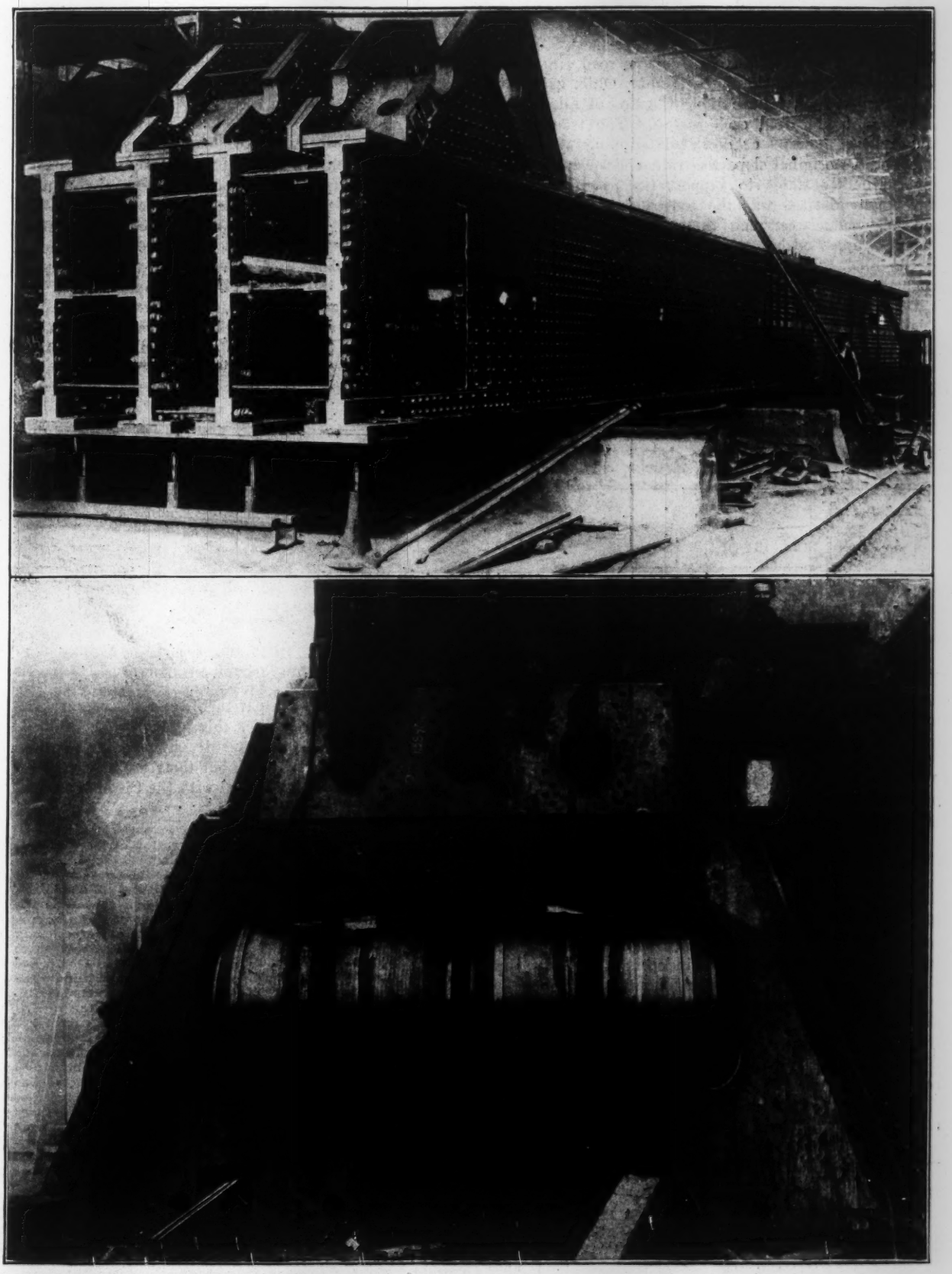
The views of two great bottom-chord sections and a pair of the huge main-shoe multiple pins are reminders not only of a pioneer design but also of the great accuracy to which the builders' shop practice was developed. So determined was the bridge company on this last point that the shop handled nothing on which coarser work was permissible; even the falsework was sublet to get it outside. The benefits of a new shop, new machines and new methods were reflected in the smooth machine-like progress of erection and in elimination of field corrections.

Taken together any collection of such views must emphasize the boldness of the erection schemes, as for the elimination of secondary stresses in bottom chords, the lavish expenditure on construction adjuncts like the 200-ft. traveler, the anchor-arm falsework, the flying bridge, the top-chord jacking links and eye-bar trusses. Yet with all this completeness of preparation, the amount of junk has been kept down to a probable 5,000 tons by studied uses of various pieces—even of the bridge itself—in different parts of the work. And finally any collection of Quebec Bridge pictures must always bring fresh to mind the names of a few men closest connected with the design and erection—Johnson, Duggan, Porter, Mitchell, Fortune; Monsarrat, Modjeski, Schneider.



COMPONENT PARTS OF THE GREAT QUEBEC

At the top, suspended span as erected on falsework near shore at Sillery, ready for transfer to six scows for floating and hoisting; view taken with erection traveler in last position. Below, the bridge seen nearly endwise from camp on north shore about two months ago. North cantilever arm completed; the south arm was finished July 28, 1916



BRIDGE ACROSS THE ST. LAWRENCE RIVER

At the top, two of the great bottom-chord quarter sections laid together in the St. Lawrence Bridge Co.'s shop for drilling, facing and matchmarking the splice. Below, the 4s-in. sleeves on 30-in. pins, in the main shoe ready to receive the first section of the great bottom chord

Concrete Side-Hill Viaduct for a Brick Highway

A side-hill section of the Dunham road, in South Newburgh township, Cuyahoga County, Ohio, gave constant trouble from failure along the line of fill, so that it had to be repaired each season. The trouble was caused by settlement or down-hill slipping of the fill triangle; the original slope was rather steep and the fill not sufficiently stable to support the pavement. The cure finally devised was the construction of a girder-and-slab viaduct carrying the pavement over the fill triangle, supported by columns along the down-hill edge of the road. This viaduct construction has just been completed.

The drawing herewith shows the arrangement and details of the viaduct construction. The columns are spaced 14 ft. on centers longitudinally of the road, and their footings are carried down 2 ft. or more below the surface of the shale, which lies practically at the original undisturbed ground level. A longitudinal girder 14x24 in. extends along the top of the columns. The raised curb of the roadway is formed integral with this girder. An 8-in. reinforced slab spans from this girder across to a bearing on the undisturbed ground near the center of the road. This bearing is made at least 30 in. wide, but locally is considerably wider. The fill is level with the down-hill curb or slightly higher, and then falls off on natural slope. No special means were employed to consolidate the fill under the roadway slab or prevent the formation of voids through settlement. The slab was, however, molded directly on this fill ground and is intended to carry the pavement independently of any support by the filed portion of the subgrade.

D. Moomaw, Road Engineer of Cuyahoga County, conceived the idea of this viaduct construction and made the design. The work extends along a length of about 275 ft. It was built by direct labor by the county road-repair department.

The following table gives the total cost and its principal elements:

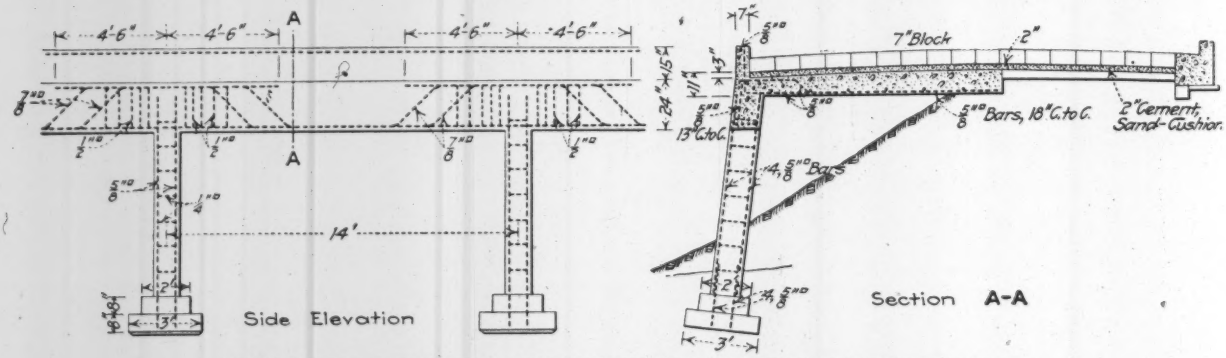
COSTS OF DUNHAM ROAD VIADUCT

Excavation, 669 cu.yd.	$484.52	$0.724 per cu.yd.
Concrete, 154 cu.yd.	1,826.68	11.86 per cu.yd.
Pavement, 327 sq.yd.	503.03	1.538 per sq.yd.
Total cost	$2,814.23	
This is subdivided into:		
Total labor cost		$1,449.94
Total material		1,056.59
Truck charges		97.50
Carfare		210.20
		$2,814.23

Items of concrete cost:

Reinforcing steel	$378.47
Cement	275.08
Sand	88.79
Slag	195.00
Lumber	21.94
Nails, etc.	1.51
Labor	529.55
Overhead (watchman, carfare off and on job, etc.)	336.34
Total	$1,826.68

A number of other pieces of road in Cuyahoga County are in a condition which it is believed requires this same treatment to effect a cure of cracking by reason of settlement of the subgrade. The general practice in Cuyahoga County roadwork is to place rod reinforcement transverse of the concrete base (or concrete pavement) where there is danger of unequal settlement by reason of (1) variation in depth of fill; (2) transition from old fill to new fill, as in widening an old road on one side and paving it; or (3) longitudinally, at points of change from cut to fill. Under the conditions prevailing on the Dunham road, however, this reinforcement is not adequate to prevent failure of the pavement, and the viaduct construction was resorted to for this reason.



DUNHAM ROAD SIDE-HILL VIADUCT, BUILT AS A CURE FOR SLIPPING FILL

Side-hill viaducts are frequently adopted in European practice on new construction, but their use as a cure for slipping fill is probably original with the Cuyahoga County authorities.

The Effect of Manganese on the corrosion of steel has recently been investigated by Sir Robert Hadfield and J. Newton Friend, and the results were reported in a paper read at the recent meeting of the Iron and Steel Institute. A number of previous investigators reached the conclusion that it is the manganese in steel, introduced in the process of manufacture, that is largely responsible for the rapid rate of corrosion of much of the present-day steel. The most notable statement to this effect was the paper published by Cushman in 1905, when he was connected with the Department of Agriculture. The series of tests carried on by Hadfield and Friend included also the effect of increasing percentages of carbon on the rate of corrosion. It was found that the addition of carbon from 0.03 to 1.63% to pure iron containing less than 0.2% of manganese produced a steadily increasing rate of corrosion as the carbon increased. These tests were made with specimens immersed both in fresh water and in sea water. When the metal was immersed in dilute sulphuric acid, there was a rapid increase in the rate of corrosion as the quantity of carbon increased until a maximum was reached at 0.8% carbon. Tests were then made on similar steels containing 0.7% of manganese. The result was a very slightly increased corrosion when the metal was immersed in both fresh and salt water, until the carbon content reached 0.4 to 0.5%. From this point upward the manganese affords a slight protection. When the steel was immersed in dilute sulphuric acid, however, the specimens containing the high percentage of manganese showed an enormously increased corrosion. Steels were then tested in which the manganese content had been increased to 2% and upward. These specimens showed greatly decreased corrodibility in neutral solution, but the corrosion in dilute sulphuric acid was greatly enhanced.

Field-Artillery Service Should Attract Engineers

By Ernest McCullough*

SYNOPSIS—The modern artilleryman is a highly trained soldier whose training combines that of every other arm of military service; the technical work of field artillery appeals particularly to engineers in competition with the engineer corps. The author gives particular attention to the system evolved for laying the fire of a battery.

Much has been written lately in the technical press on the importance of engineer troops in war, and engineers in civil life have been shown the work of engineers in war. Apparently it has been overlooked that the artillery, particularly field artillery, is an arm especially appealing to an engineer.

The modern field artilleryman is trained in every arm of military service. He is drilled in foot movements like the infantry, the new regulations coördinating the artillery foot drill with infantry drill. He receives the same instruction in equitation and hippology, the riding and care of horses, as that received by the cavalrymen. He is trained in the use of the yellow and red semaphore signal flags and in the use of the white and red flags for signaling with the International Morse code; in the care and use of field telephones and the care and use of wireless. He learns various other methods for quick communication, and his instruction in this respect is, for all practical purposes, as good as that of men in the signal corps. He becomes skilled in scouting, map reading, road and panoramic sketching. Besides this, he learns to fire the pistol and the field gun. Not only must he know how to fire the gun, but he must know the name of every part of it, and the limbers, caisson, battery wagon, store wagon, forge wagon and harness. He must have also some knowledge of the construction of field fortifications and demolition of structures.

The civil engineer will be interested in the surveying and the firing problems. The mechanical engineer will be interested in the splendid machines he has to handle. The electrical engineer will be interested in the telephones and the wireless apparatus, the construction engineer in the camps and equipment. Military work is to the chief-of-staff a science; to the officer in the field with troops it is an art; to the men in the ranks it is a trade. The artillery arm has so many trades not strictly military connected with it that the engineers of the country should look carefully into the claims of the artillery before deciding to join the engineers.

Development of Field Artillery

The types of guns used in the Civil War were rigid and had to be returned to position and "relaid" after every shot, for the recoil sent them back several feet. Barring the universal substitution of breech-loading guns for the old muzzle loaders and an increase in power, there was little advance in field artillery up to 1892. In that year General Langlois, of the French army, wrote a book giving a history of artillery from the earliest times, in which he set forth specifications for the field gun of the future. The French general staff got busy and finally in 1898 produced a gun that could be used for "indirect fire" in a far more perfect way than any previous gun, for indirect laying had long been the subject of experiment. In the new gun the recoil was controlled and the gun did not "kick off" the target, but was held to the previous laying after each shot. Then the breech mechanism and the ammunition were so improved that the gun become a rapid-fire gun, which could deliver ten aimed and twenty unaimed shots per minute.

Field artillery has one work to perform. The training of the officers is practically identical in all the schools, no matter what the language spoken by teachers and students. General Langlois in 1892 presented his specifications, and ordnance officers everywhere turned their attention to the design and manufacture of guns to fill the specifications. The field-artillery officers gave the *matériel* a complete try-out in the field, and the modern gun was thus evolved. Proceeding from the same starting point, equipped with the same knowledge, learning through various channels of the work done by other men and given opportunity to view the completed product as finally adopted by other countries, it is not strange that the light fieldpiece is practically standardized the world over.

In 1898 the United States adopted the 3.2-in. gun (called the "three-point-two"), which embodied some of the features deemed by General Langlois to be essential, and finally in 1902 adopted the 3-in. field gun. The breech mechanism was brought to its present satisfactory state in 1905, so that the American 3-in. field piece is practically the equal of the famous French 75-mm., the *soixante-quinze* idolized by the French nation. The bore of the "75" lacks only 0.05 in. of being as large as the 3-in. gun, which is the caliber of the light field gun of the United States, Great Britain and Russia. The caliber of the Austrian light fieldpiece is 3.01 in. and of the German 3.03 in. The light fieldpiece of the remaining eighteen powers that have armies large enough to be considered as important have a caliber of 75 mm. (2.95 in.). The weight of the projectile differs very little, that of Great Britain being 18 lb., of the United States and Germany 15 lb., and of France 15.96 lb., the weight being less in all the other countries.

Field artillery is classed as horse, light and heavy in all countries other than the United States. Here we have only horse, light field and siege artillery. Actually we have no siege artillery in the United States, the pieces so classed coming under the designation of "heavy field artillery" in other countries. Faulty phraseology and careless diction placed the United States in a rather ridiculous position in this classification, but if the present "preparedness" campaign accomplishes the hoped-for results we may have some siege artillery. Lack of reasonable mobility places artillery in the siege class, and

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the line is fixed above the 4.7-in. gun and the 6-in. howitzer, these being heavy field artillery and the lightest equipment sometimes classed as siege artillery.

A fieldpiece fires at an elevation of less than 20°, a maximum of 15° with the end of the trail on the surface, additional elevation for distant ranges being secured by sinking the trail in a trench. A howitzer fires at elevations of from 20° to 40°, a mortar at from elevations of from 40° to 70°—the elevations referred to being the angle above the horizontal through the axis of the bore.

The howitzer is shorter than the field gun of practically equal mobility, so that the 3-in. field gun and the 3.8-in. field howitzer are paired in infantry divisions of a mobile army as light artillery. The pieces are hauled by six-horse teams and can readily accompany infantry anywhere. Only the commissioned and noncommissioned officers and privates of special details are mounted. The rest of the enlisted men accompany the batteries on foot, mounting on the limbers and caissons and on axle seats of the pieces for rapid movements over short distances.

When every man is mounted, the 3-in. battery becomes horse artillery and accompanies cavalry, having the mobility of that arm. No field howitzers accompany cavalry. The 3-in. field gun fires a 15-lb. projectile, and the 3.8-in. field howitzer fires a 30-lb. projectile.

The 3.8-in. field gun is a heavy fieldpiece hauled by a team of eight horses. For short distances only, the gun is capable of rapid movement—not being so mobile as the lighter piece. It fires a 30-lb. projectile. The field howitzer of equal mobility paired with this gun is the 4.7-in., firing a projectile weighing 60 lb. The reader will notice that in every case the weight of the projectile, consequently the power, of the howitzer is in each case double that of the field gun with which it is paired. The 4.7-in. gun fires a projectile of 60 lb., and the 6-in. howitzer one weighing 120 lb. The 3-in. fieldpiece is the weapon used against infantry and cavalry operations, and the 3.8-in. howitzer gets into action when the attackers intrench. Heavier pieces are brought into service as the protection of the enemy by intrenchment assumes a more permanent form.

Laying the Fire

Modern artillery fires whenever possible from concealed positions and it is with this phase of the subject that the artilleryman has use for his mathematics. When indirect laying became a subject of interest to artillery officers, all the work was trigonometrical and the operations involved partook of the nature of careful triangulation work. They were slow and required some degree of education. It was essential that the ordinary enlisted man be able to understand how to lay a gun properly, so the French artillery appointed a board of officers, who had been for years working on the new *matériel,* to get up a simple system for laying the gun when the target was invisible.

In 1897 they reported in favor of a new graduation of the circle whereby every element in the data pertaining to the delivery of fire became a function of the range. With sights on each piece graduated horizontally in "mils" and with range quadrants on each piece graduated vertically in the same system and with all observing instruments similarly graduated, the idea met with an enthusiastic reception. It is now used the world over by field artillerymen.

A mil is the one-thousandth part of the range, approximately. For the purposes of delivery of fire it is to all intents and purposes exact. The ratio of the radius of a circle to the circumference is 6.283185; and if the radius is divided into 1,000 equal parts, there will be 6,283.185 such equal divisions on the circumference. The artilleryman divides the circle into 6,400 parts, because this number can be halved or quartered and continually subdivided by 2 to any limit without a fractional remainder. The error involved is small from the standpoint of the mathematician, and from the standpoint of accuracy required in the delivery of fire there is no error at all. The sixty-four-hundredths part of a circle is called a mil (millième) because it is the one-thousandth part of the radius, or, for artillery fire, the range. It is an angle whose natural tangent is 0.001.

1 mil = 3.375 min., exactly
0.3 mil = 1 min., approximately
1° = 18 mils, approximately

To convert degrees into mils the writer in reconnaissance work with a common compass subtracts mentally 11% from the number of degrees and multiplies by 20 to obtain the mils. Never subtract fractional degrees; but when fractions occur, take the next higher unit. For example, the number of mils in 90° = $(90 - 10) \times 20 = 1{,}600$. The number of mils in 45° = $(45 - 5) \times 20 = 800$. The quarter-circle is graduated into 1,600 mils, the eighth-circle into 800, the half-circle into 3,200 mils, etc. Possibly before the next war the United States may issue to the artillery prismatic compasses with the mil graduation. In fact, there exists no good reason why all officers in the army should not have such compasses, which would have the usual letters designating the cardinal points.

The methods of laying, about to be described, have been supplemented by a method, described recently in an English artillery magazine, in which compasses are used. The United States Army School of Fire for Field Artillery issued a clearly written pamphlet on the subject in January, 1916. Experiments indicate that a transit-compass needle is not materially affected by large masses of metal at distances greater than 50 ft.

The artilleryman in discarding sines, tangents and other functions of an angle went to the astronomer to obtain a term to use for expressing the angle subtended, at the point observed, by the platoon front. The term used is "parallax" and is expressed in mils. It is quickly obtained by dividing 20 by the number of thousands of yards in the range, the width of a platoon front—that is, the distance center to center of bores of two adjacent pieces—being 20 yd. Thus if the range is 2,800 yd., the parallax $= \frac{20}{2.8} = 7$, for fractions are disregarded. The work is done mentally in the field, and $20 \div 3 = 7$ (approximately) would be the result of the brainwork of the observer.

This disregard of fine results troubles an engineer more at first than any other one thing in artillery work. However, all that is required at the first salvo is to bring the "burst" within the range of observation of the field glasses and battery commanders' telescope, after which corrections are applied in accordance with the laws of probabilities, provided no serious errors of range have been introduced. Since 1 mil = 3.375 min., the disregard of 0.4 mil will make a difference of less than 1 yd. in 1,000. This is a far greater degree of accuracy

than the best guns possess, accurate though the modern fieldpiece may be.

The mil measures chords of angles for given radii, and at 1,000 yd. an angle of 1 mil subtends a chord of 1 yd.; at 3,000 yd. a chord of 3 yd., etc. If a house standing normal to the line of sight from a battery has a length known to be 60 ft. (20 yd.) and the observer at the telescope measures an angle of 25 mils from the rear to the front corner, the range $= \frac{20 \times 1{,}000}{25 \text{ mils}} =$ 800 yd. Field glasses and telescopes are made for use by artillerymen, having an interior glass diaphragm engraved with vertical and horizontal lines similar in principle to stadia wires. The interval between the lines being known, angular measurements in mils are quickly made or estimated; and when the distance between objects is known, the range may be quickly estimated. This range is set off on the gun and shots fired. The burst is observed through the glasses and corrections made by rounds, or salvos, until the object is bracketed. Such firing is termed "fire for adjustment," and the adjusted fire is termed "fire for effect."

The battery commander sets up his telescope, whenever time permits, on a flank of the battery and at some little distance from the gun on the right, which is known as the right or No. 1 gun. If the gun is in a position where the target can be seen, it has been customary to use direct laying, in which the gunners sight on a portion of the target assigned to them. A battery of fieldpieces is a power plant, and the battery commander is the superintendent of the plant. In order to have it completely under control he may, even against visible targets, use indirect laying, in which the gunner sights on an aiming point and sets off a deflection to the target announced by the executive officer. It is exceptional to use direct laying, for every effort is made to conceal the positions of guns, the battery commander, assisted by observers, giving all necessary data to the men at the guns. It is mainly with indirect laying that mathematics enters into the matter.

Fig. 1 illustrates the geometrical truth that when two parallel straight lines are intersected by a straight line

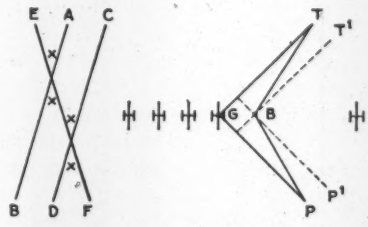


Fig. 1

Fig. 2

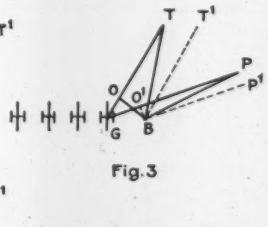


Fig. 3

the alternate angles are equal; thus the angles marked *X* are equal. This principle is used in what is called the "parallel method," a method so simple that any enlisted man can lay a gun. For the enlisted man the method is partly graphical. In Fig. 2 let *T* be the target, *P* the aiming point, *B* the observing station, and *G* the right gun of a battery. The distance from *B* to *G* will be measured, estimated or guessed, if *B* is exactly in line with the guns. If it is a little in front or behind the guns, then the distance is taken to the line *TG*, or this line produced. The range from *G* to *T* is measured, estimated or guessed. The offset distance in yards from *B* to the line of fire from the gun to the target is divided by the thousands of yards in the range from the gun to the target, and the angular result in mils is measured from the line *BT* to the line *BT'*.

The telescope is pointed to the aiming point and similar computations and measurements made to the line *BP'*. The angle *T'BP'* is measured and the result given to the gunner on the right piece. He sets his panoramic sight at the angle given, and the trail is moved to the right or left until the vertical wire is on the indicated aiming point. The axis of the gun then lies in the line *GT*. In Fig. 3 the aiming point is in front of the battery. An officer of experience and a man familiar with algebra will not take the time to set auxiliary aiming points on the lines *BT'* and *BP'*, but will compute the angle of deflection by using positive and negative signs in connection with the angles subtended. This is known as the parallax method, formerly used altogether.

The writer will not describe the parallax method further than to say that, the parallax for a platoon front having been obtained, the distance from *B* to *G* is given in terms of platoon fronts and the angle *BTG* is found by multiplying the parallax by this number of platoon fronts, for the parallax is the angle subtended by a chord of 20 yd.

The angle *PGT* is the deflection of the right piece only. If the other guns set off this deflection, with their panoramic sights on the aiming point, the fire would not be directed on the target by all the guns unless the target was quite extended. It is therefore necessary to find the convergence difference in order that each gun will have the necessary deflection to fire directly at the target. Obtain the parallax of the target and give it the positive (+) sign. Similarly obtain the parallax of the aiming point, the sign of which will be positive when the aiming point is in front and negative when the aiming point is in the rear. The parallax of the target is subtracted algebraically from the parallax of the aiming point to obtain the "convergence difference."

Example—The deflection of the right piece is 3,320 mils. The parallax of the target is + 7 and the parallax of the aiming point is —6. The convergence difference $= -6 -(+7) = -13$. The deflections then become 3,320, 3,307, 3,294 and 3,281, for the convergence difference is applied progressively. These deflections being set off and fire opened, each shot should strike in the same place, provided the range is correct, the data properly computed and the *matériel* is perfect.

Assuming that the front of the target subtends an angle of 60 mils and it is desired to distribute the fire over the whole front, divide the front of the target in mils by the number of guns firing to get the "distribution difference." This for the foregoing example is $\frac{60}{4} = 15$ mils. Adding progressively, the deflections become 3,320, 3,322, 3,324 and 3,326. The distribution difference is always additive. The algebraic sum of the convergence difference and the distribution difference is the "deflection difference."

Having obtained the deflection and the deflection difference, and the panoramic sight of each gun being set on the aiming point, the battery commander can by changes in deflection, or deflection difference, open and close the "sheaf of fire" so as to distribute or concentrate the fire of the guns at will. The cannoneers do not see the target

and cannot know the effect of the shots. The observers on high points communicate by telephone, flag or other signal system the result of each shot to the battery commander, who alters his data as required. The battery commander is presumably seated on the upper end of a very light ladder in rear of the battery, the better to observe the fire and direct his power plant. He is a good target for sharpshooters, but then someone has to take some risk in battle.

The path of the projectile is a parabola, and the battery in action may easily be likened to several fire engines at work with the chief of the fire department in the place of the battery commander. The stream of water has a path of travel of the same form as that of the projectile. The parabola for all practical purposes of the field artilleryman is rigid—that is, the range is the shortest distance from the muzzle of the gun to the target, whether it is horizontal or inclined. The errors liable to arise because of this assumption are well within the limit of errors that may be caused by inferior *matériel* or errors in the data. The first shots never hit a target, except by accident, but they get so close as to make the target feel very uncomfortable and anxious to seek cover. Corrections are rapidly made, and the data are set off on the sights so that the following shots get closer to the target.

In the example given, the captain may have observed that the front had not been estimated correctly and it was necessary to narrow the fire. His command may be "Close by 10"; then the gunner on the first piece makes no change, the gunner on the second subtracts 10 mils and his deflection becomes 3,312, the third gunner subtracts 20 and his deflection becomes 3,304, the fourth gunner subtracts 30 and his deflection becomes 3,296. The deflections are set off on the sights and the trails are swung or the traversing gear operated to bring the vertical mark again on the aiming point, when the guns will be pointed properly to place their fire where wanted.

If the command had been "Open by 10," each gunner except the first would have added 10 mils multiplied by the number of his piece counting from the first piece.

If the captain observed that all the shots were falling, say, 40 mils to the right, he would command "Add 40," and each gunner, including the gunner on No. 1 piece, adds 40 mils and brings his aiming point on the vertical mark. Observing that the fire is falling 40 mils to the left the officer commands "Subtract 40," whereupon every gunner subtracts 40 mils from his deflection.

These data are kept on small cards by chiefs of section, the sergeants. The chiefs of platoon, the lieutenants, cannot use paper and pencil, but must do all work mentally. Nevertheless they must remember the data. It is thought speed is sacrificed if time is taken to write down data.

The angle of site is the angle of slope of the range in mils. It is set off when the range is set off, in order that the axis of the bore will lie in the trajectory of the projectile. The corrector reading is the number of mils set off on the fuse setter to insure the fuse burning until the shrapnel reaches the proper point on the trajectory. The fuse setter is an attachment on the rear of the caisson, consisting of a movable ring graduated for range. The range being ordered, it is set off on the fuse setter and the corrector correction applied. When the corrector reading is 30 and the range is correct, the shrapnel will burst at a height of 3 mils above the ground at the end of its flight. When shots are "over" or "short," slight changes are made in the corrector setting or in the angle of site, or both, together with changes in range until the shots reach their mark.

Masking the Fire

It is necessary to fire over a mask, for concealment, which is known as "defilade." Dismounted defilade is when a man standing at the gun can just see over the mask and observe the target. In mounted defilade a mounted man at the gun can just see the target over the mask. The gun is perfectly concealed with a "flash" defilade when the flash of the gun is concealed. A defilade of 4 yd. is sufficient to conceal the flash of a gun by day. A flashless powder is now made. On a fairly uniform reverse slope a flash defilade may be quickly obtained by posting a dismounted man in the position of dismounted defilade and in his rear a mounted man where mounted defilade will be obtained. The guns are then posted in rear of the mounted man at a slightly greater distance than that separating him from the dismounted man.

It is necessary to know whether the shot will clear the mask. Let y = height of mask in yards, x = distance from gun to mask, and $R - x$ = range from mask to target in hundreds of yards. Then $x = \frac{4y}{(R - x)}$.

Example—The range from mask to target is 2,000 yd., height of mask 15 yd.; $x = \frac{4 \times 15}{20} = 3$ (300 yd.).

By consulting a range table it will be found that the angle of elevation for a range of 2,000 yd. is 52.4 mils, and the angle of site to the top of the mask is $\frac{15}{3} = 50$ mils, so the shot will clear the top with a small margin. The margin is so small that the mask is a poor one, for it is unnecessarily high. The entire distance from the target to the mask is dead land and perfectly safe. The guns should be close enough to the mask to afford perfect concealment and yet be able to depress the muzzles and cover a considerable length of the advance from the target, which is usually occupied by troops. This particular mask should be not over 12 yd. high. To fix the proper height of mask, use the formula $S = 30 (K - 1)$, in which S = angle of site in mils and K = range in thousands of yards.

Let the range from the gun to the target be 2,300 yd.; then $30 \times (2.3 - 1) = 39$ mils, which upon reference to a range table is the angle of departure for a range of 1,350 yd., so that the danger zone from the first position of the target will be 950 yd. and the enemy will have to approach within 1,150 yd. of the mask before the gunfire fails to reach them. At this range the infantry can handle the enemy with machine guns and rifles. With a lower mask, say of 5 yd., the danger zone may be greatly widened.

Readers who may have become interested in the work of field artillery through the reading of this sketch should procure "Notes on Field Artillery," by Capt. Oliver L. Spaulding, Jr., or "Elements of Modern Field Artillery, United States Service," by Maj. Harry G. Bishop, or both books. The authors were lecturers on the subject at the Army Service Schools, Fort Leavenworth, Kan., and the books contain the substance of their lectures.

Old Iron Bridge Damaged by Electric Car

The damage done to a bridge truss by the derailment of an electric car is shown by Fig. 1. The track is close to the truss, so that the blow was almost longitudinal, and the car evidently struck the truss post with its roof and body. The accident occurred recently on the middle span of the Galena Ave. bridge over the Rock River at Dixon, Ill.

The post is the second from the south end of the east truss of the middle span. The channels and lacing bars are bent and buckled, showing the toughness of the metal, but the rivets are tight and only one of the lacing bars seems to have broken, the fracture occurring at the rivet hole. The post is about 8x12¼ in., being composed of two 8-in. channels. For temporary repairs a two-pile braced bent was placed under the panel point, and blocking was wedged up against the foot of the post, while a timber post was placed upon this support and wedged up against the top chord. The bridge carries two tracks, but one track (and half the roadway on the damaged span) is now closed to traffic. The damaged post was cut away by torches and a new post put in place, the repairs being made by the American Bridge Co.

The Galena Ave. bridge is peculiar in that, while it was built 30 years ago for wagon traffic, it now carries a double-track electric railway for both city and interurban cars. The railway line is single track, but presumably a track in the middle of the bridge roadway would have interfered with street traffic, which is an important consideration as this is the only highway bridge across the river at Dixon. It was built in 1885 by the Milwaukee Bridge and Iron Works, of Milwaukee, Wis.

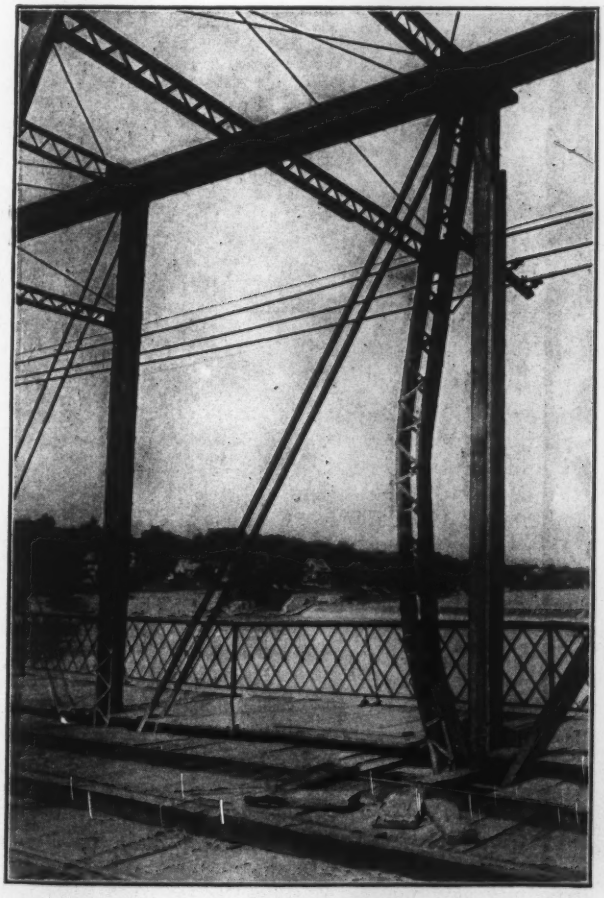


FIG. 1. DAMAGED TRUSS POST OF THE GALENA ST. BRIDGE AT DIXON, ILL.

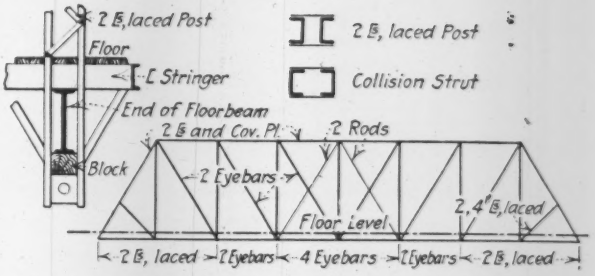


FIG. 2. GENERAL DESIGN OF TRUSS SPANS OF THE GALENA ST. BRIDGE AT DIXON, ILL.

The bridge consists of five through-truss spans of 175 ft. The design is shown in Fig. 2. The end posts and top chord are composed of two channels and a cover plate, with lacing on the under side. The end posts have 14¼-in. channels and 16-in. plates. The bottom chord for two panels at each end is composed of two channels with top lacing; the next panel has two eye-bars, and the two middle panels have four eye-bars each.

The truss posts are pairs of channels with flanges outward, and laced, as shown: First post, 6-in. channels, 11½ in. over backs of channels; second post, 8x12½ in.; third post, 7x12 in.; middle post, 5x11 in. At the bottom of the post a saddle is riveted between the channels to carry the end of the floor-beam, and through this saddle passes the pin connection for the diagonals.

The collision struts are 4x10 in., with two 4-in. channels having the flanges inward. The diagonals are pairs of eye-bars, about 3½x1 1/16 in., 3x1⅛ in. and 2½x⅝ in. in the three panels. The counters in the two center panels are about 1-in. rods, with turnbuckles. At each panel point there are a top strut and sway bracing. The metal of the trusses seems to be in good condition and is well painted.

The bridge originally had wooden floor-beams and stringers. When the electric railway was laid across it, about 12 years ago (according to our information), the floor system was rebuilt. There are now plate-girder floor-beams on which are 13 lines of channel stringers with wooden spiking strips. The two channels under each rail are placed back to back and have riveted supports for a timber stringer between them. Each sidewalk has four lines of channels. The flooring is of transverse planks. There are no curb or wheel-guard timbers. The longitudinals in the view are to keep traffic to one side of the roadway of the damaged span. The rails are of tram-head section.

It is stated that, when the changes were made, calculations showed that the bridge was strong enough to carry the increased loads. Under the ordinance permitting the railway to use the bridge the company was charged with the duty of putting and maintaining the bridge in proper condition.

The railway company states that while the bridge has two tracks (for the traffic reasons noted above) there is a rule by which cars are not allowed to pass on the bridge.

Field and Office

6,000-Ton Rock Slab Pinned in Place To Save Bridge

By W. H. Warnecke*

An almost separated mass of rock weighing about 6,000 tons, on the face of a steep cliff overhanging the West Shore R.R., was recently pinned to the sound rock of the cliff by steel pins as a precaution against its sliding and wrecking a railroad bridge. The following paragraphs describe the work done to secure the slab.

The cliff in question (Fig. 1) is located immediately south of the Highland Falls station on the West Shore R.R., now called the River Division of the New York Central R.R. The rock is granite of an inferior grade, with nearly vertical stratification. The partly separated mass is about 200 ft. long, with an average height of 100 ft., and from 5 to 8 ft. thick. It starts from about midway between the upper and lower chords of the bridge shown in Fig. 1, the greater portion of the mass being wholly above the bridge.

The bridge seen in the figure is one of two built in close proximity to each other in this locality, both of the bridges being placed close to the precipitous faces of the rock cliffs forming the shore. These bridges are peculiar in that they were constructed to carry the tracks over transverse clefts in the rock bottom of the Hudson River, which it was found impossible safely to fill on account of the steep slope of the rock toward the center of the river. Considering the conditions, it was clear that any attempt to remove the loosened mass of rock would prove extremely dangerous to traffic. Therefore, a scheme for supporting the slab was adopted. Ten holes, 11¼ in. in diameter and from 11 to 20 ft. apart, were drilled from 11 ft. to 15 ft. into the rock at an angle of 60° with the vertical at the level of the upper chord of the bridge shown in Fig. 1. A gas-driven core drill (Dobbins, Class H-G) was used in drilling the holes. The drilling outfit is shown in Fig. 3, mounted on the platform built on the upper chord of the bridge. This view also shows a completed hole with a piece of casing projecting from it, and near-by is a piece of the core removed from the hole. Owing to the rough face of the rock, it was necessary to facilitate the starting of the drill by building a wooden box 24 in. square, the box being made fast to the rock by means of angles and expansion bolts, and cementing in place in the center of the box a 12-in. nipple 12 in. long. This nipple assisted in keeping in place the steel shot used for cutting the rock. The starting of the drill was thereby simplified and its penetration into the rock greatly augmented. The seam between slab and cliff was successfully passed, though some shot was lost.

Upon the completion of the drilling of the holes, 10-in. solid steel pins, made in various lengths to suit the respective lengths of holes, were put in place, to serve as support for the hanging rock. Fig. 2 shows the operation of placing one of these pins. One end of the pin was raised to the level of the hole at the face of the rock, and the other end was then raised until it had the same inclination as the hole. In this position it was a comparatively easy matter to slide the pin into place. A small derrick resembling a gin pole was used.

*Engineering Department, New York Central Railroad, Poughkeepsie, N. Y.

FIG. 1. CLIFF WITH HANGING ROCK, HIGHLAND FALLS N. Y.

FIG. 2. PINNING THE ROCK DOWN WITH 10-IN. STEEL PINS

FIG. 3. CORE-DRILL OUTFIT ON BRIDGE, DRILLING HOLES FOR THE PINS

The space surrounding the pins was filled with rich cement grout by means of a ¾-in. pipe with a hopper on one end, the pipe being long enough to reach to the bottom of the deepest hole. The pins were cut 1 in. longer than the length of the respective holes, causing them to project 1 in. beyond the face of the rock. These projections were treated with coal-tar pitch and covered with a rich cement mortar applied to woven-wire mesh fastened to ½-in. steel bars of U shape. These bars are held in place in a horizontal position at the centers of the pins by expansion bolts inserted into the rock 3 in. on either side of each pin. The surface of the mortar is also covered with hot coal-tar pitch.

For auxiliary support the foot of the loose slab was underpinned with timber struts. A shelf was cut into sound rock a little above the floor level of the bridge. Yellow-pine struts, 12x12 in. in section and 3 ft. apart, were then inserted under the projecting slab and held in place by expansion bolts set into solid rock. The work was carried out by close coöperation between the Engineering Department and Maintenance-of-Way Department; G. W. Kittredge, Chief Engineer; J. W. Pfau, Engineer of Construction; R. E. Dougherty, District Engineer; George W. Vaughan, Engineer Maintenance-of-Way; and H. C. Thompson, Division Engineer.

Keeping a Cliff from Falling on Railway Tracks

A good-sized job of geological dentistry is being performed by the Pittsburgh division forces of the "Panhandle" railroad (Pittsburgh, Cincinnati, Chicago & St. Louis Ry.) in plugging up the cavities in the face of Mount Washington, across the Monongahela River from Pittsburgh. The weathering of this cliff has been a constant source of trouble and maintenance expense ever since the toe of the slope was cut away to make room for the two tracks of the Pennsylvania low-grade freight line detouring around the heart of Pittsburgh.

Immediately above the track level there is a stratum of hard clay, varying from 15 to 30 ft. in thickness, underlying the ledge rock. As the clay sloughs away under the action of the elements, it becomes necessary to provide support for the overhanging mass of rock above it. In the past nine years over 800 ft. of wall has been built for this purpose, and the present season's work will add 300 ft. to this. Similar support is also to be furnished some rock ledges farther up the cliff.

The work is being done by company forces and requires the placing of a sloping concrete wall from firm rock below the track level to the under side of the rock mass above. It was planned to handle the work from a portable tower, but as this could not be obtained in time an inclined skidway was built up the side of the hill. Concrete is hoisted in a wooden car, sliding on skids, and dumped into a hopper at top, from which it is chuted to place.

SUPPORTING THE OVERHANG OF MT. WASHINGTON BY CONCRETE TOE-WALL

The work is being done by the Panhandle railway. Forms are in place for the wall, and a spout from a distributing hopper supplies the concrete

Cutting Clay Tunnel Face with Special Knife

Since the ingenious application of a modified drawshave or drawknife to cutting clay in tunnel work in the St. Clair River tunnel at Port Huron, Mich., over twenty years ago, this tool has been standard for use in similar operations in the clay ground of the lake plains. Two tunnels now in course of construction at Cleveland are excellent illustrations of the use of the drawknife. In the course of the work a change has been made in the tool, and at present both the simple and the improved form are in use on the same job. The drawknife as generally used up to the change of design is as shown in Fig. 1. The steel of the blade is about 1¾x¼ in., bent hot to the roughly semicircular shape indicated in the sketch and turned at the ends to form eyes. Sawed-off spade handles are inserted in these eyes. The lower edge of the knife is ground on its inner face to a sharp edge and is reground here as it becomes dull, until the blade finally becomes so narrow that it is too weak to use. In new condition the failure is apt to be in the handles; but as the blade wears narrower, breaks in the steel are likely to occur. Great muscular effort is required to use such a knife with full efficiency. Two men usually work on one knife, with a third giving a hand in entering the blade and later receiving the slab of clay cut down. Some suggestions were made for a U-shaped bar handle in place of the two straight handles, this bar handle to be made of strong pipe, which would not only give more convenient hold but also eliminate breakage of handles. The final design of John W. Dolman, engineer in the City Water Department is shown by Fig. 2. At first a handle length (offset length) of 18 in. was thought best; but it developed at once that this was too long for most effective manipulation, and it was shortened to 12 in., as indicated in Fig. 2. Fig. 1, redrawn from a sketch made from life, illustrates the typical manner of using the drawknife as practiced in the Cleveland tunnel work. This sketch was made in the West Side lake tunnel—the same in which the disastrous accident of July 24, 1916, occurred, but in the land heading. In this heading, knives of both types shown in Fig. 2 are used side by side, although the old style is the regular equipment. The experience here has amply demonstrated the greater robustness and generally greater serviceability of the new knife.

FIG. 1. HOW THE DRAWKNIFE IS USED ON A CLAY TUNNEL FACE

In the Central Ave. tunnel being driven under the Cuyahoga River for a water-main connection from the east to the west side, a job now completed, these knives were also used.

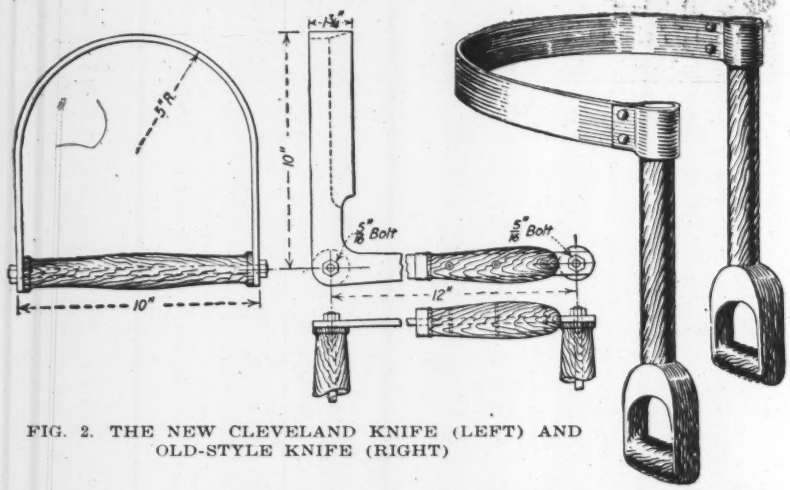


FIG. 2. THE NEW CLEVELAND KNIFE (LEFT) AND OLD-STYLE KNIFE (RIGHT)

The lake tunnel is under Gustav C. Van Duzen as Superintendent, while the river tunnel is handled by Charles Vandevelde. John W. Dolman is in charge for the Water Department, under the direction of C. P. Jaeger, formerly engineer and now commissioner.

Sandbag Dam Saved by Canvas Cover

Emergency work was necessary to save the water-supply of Charleston, S. C., at the time the Goose Creek dam was breached by flood, as reported in *Engineering News* of Aug. 3. The ocean tide comes up to the dam with 6-ft. rise, and there was risk of salt water backing up into the reservoir. Fig. 1, showing the breach on Sunday, July

FIG. 1. BREACH OF GOOSE CREEK DAM
West wall of west spillway at right

16, 1916, the day after the washout, suggests how readily this might have occurred. Closure of the breach succeeded in three days, although the gap was 100 ft. wide and over 13 ft. deep.

The embankment (the grassy surface seen in Fig. 1) was cut through late Saturday, July 15, by water overflowing about 14 in. deep. Material and tools could not be assembled on the ground for two days, however. On

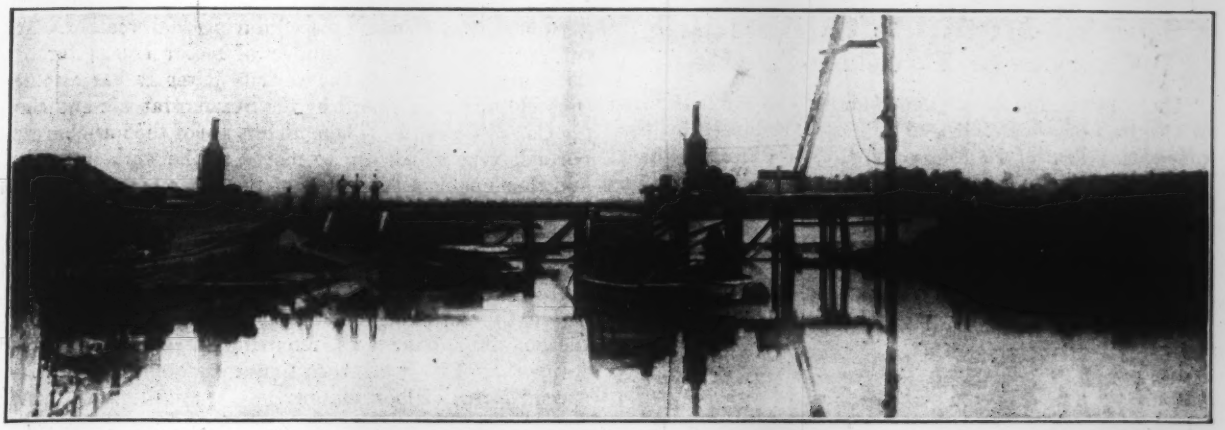


FIG. 2. SANDBAG EMERGENCY DAM (IN REAR) AND PILEDRIVING FOR CRIB COFFER-DAM OF PERMANENT CLOSURE

Tuesday morning work was begun on throwing a sandbag dam across the break; operations were in charge of W. R. Bonsal, vice-president of the Seaboard Air Line, working with a railway wrecking crew. The sandbag dam was completed early the next morning.

Low tide setting in soon afterward, the overflow over the bags began to disintegrate the temporary structure, and it was apparent that some means of protection had to be found. The expedient was tried of covering the worst places with canvas, weighted down on the upstream side and lapped over the dam. This proved successful, and the whole dam was then covered similarly.

This temporary dam was about 25 ft. wide on the bottom, 6 ft. wide on top and 120 ft. long. Water flowed over it 6 in. deep at the maximum.

Following the successful emergency closure of the breach, permanent repairs are being made. Heavy piling was driven all the way across the breach and into the embankment on both sides. Three rows of piles were set, the two upper rows about 8 ft. apart with piles spaced 8 ft. on centers. On the upstream side a line of 4-in. sheetpiling was driven, and the space above and below the sheetpile wall was filled with clay bags. In Fig. 2 (this as well as Fig. 1 is from J. W. Ledoux, of Philadelphia) the pile wall is shown practically completed, with the sandbag dam back of it still overflowing. This view was taken Saturday, July 22.

A fire engine obtained from the City of Charleston then served as means for making hydraulic excavation of clay from high clay banks near the breach and sluicing this clay into the pile dam closure. The dam as a whole, 2,300 ft. long, will be provided with larger spillways.

Home-Made Horse-Operated Wagon Loader

The accompanying views show a rather crude wagon-loading device, but at the same time an improvement over the cruder methods often met with at small gravel pits. It is in use in New Mexico, where horse power is cheap. The designer is A. E. Saint Morris, County Highway Director of Valencia County.

The gravel loader consists of an elevated bin, holding approximately 6 to 8 cu.yd., with an inclined floor and three unloading spouts extended over the wagon way. An elevating hopper is provided to raise the gravel to the bin from a pit in the rear. This device is made entirely of wood. The arms are round poles braced with planks. The hinges are merely round iron pins through the poles, and the bearings are holes bored in the square timber supports. The motive power for raising the loaded hopper is two two-horse teams.

The hopper is loaded from a platform on the edge of the gravel pit, which is provided with an opening and chute discharging into the hopper, when in its lowest position. Gravel is brought to the platform by fresno scrapers. The same two teams are used for the fresnos.

The operating force at the pit consists of only the three men shown in the view. One man shovels the gravel from the platform to the elevating hopper. The other two handle the teams. There is no waiting of wagons to load. The teamster does his own loading by opening the gates of the bin chutes. With teams at $3.50 to $4 per day, the cost at the plant shown is about 20c. per cu.yd. for excavating and loading the gravel in the wagons.

HOME-MADE WAGON LOADER, VALENCIA COUNTY, NEW MEXICO

Special Methods in Building Concrete Footing Walls

The construction of concrete foundations for buildings is the line of work followed almost exclusively by the contracting firm of F. Johnson & Co., of Chicago, which subcontracts this work from general mason contractors. In making this a specialty various methods have been introduced to increase the efficiency and economy of operation, and employees are encouraged to offer suggestions in the way of further improvement.

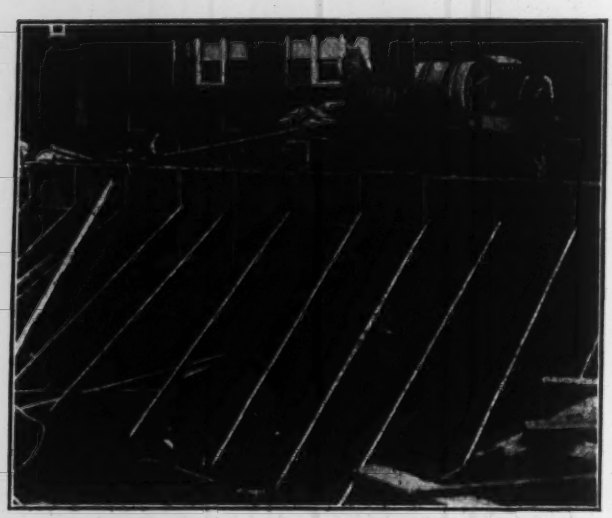


FIG. 1. SECTIONAL WOOD FORMS FOR CONCRETE FOUNDATION WALLS 9 FT. HIGH

Among the important methods are: The use of large motor trucks for rapid delivery of materials, simple sectional wood forms that can be used many times and require very little lumber or bracing, placing the concrete with a minimum amount of handling, and the use of non-freezing compounds in winter in such a way as to insure good quality of concrete placed during cold weather. The company owns a large number of motor trucks for distributing materials to the various jobs. It has also ten concrete mixers, all built by the Standard Scale and Supply Co., of Chicago. These mixers are of the low-charging portable type which can be moved as the work progresses and thus enable a large part of the concrete to be spouted directly into the forms.

Sectional wood forms are used for practically all basement walls 9 ft. or less in depth. These forms vary from 3 ft. 6 in. to 4 ft. 6 in. in width, and from 4 ft. to 9 ft. in length, depending upon the requirements of the foundation. For basements requiring a greater depth of forms than 9 ft. it has been found preferable to construct special forms of loose lumber.

When using the built-up sectional forms, the concrete footings are first put in. The forms for the outside of the wall are then set up, but before erecting a form section, strips of No. 28 hoop steel are nailed at the bottom of the frame. These are about 1 in. wide and of sufficient length to nail to the outside of the frame of the outer form, extend across the wall and permit nailing to the inside of the frame of the inner form. When the inner forms are set up, by placing wood wall gages between the side forms at the bottom, drawing the steel hoops taut and nailing them on the inside of the frame of the inner form, the lower parts of the forms are securely held on the footing. At intermediate points between the bottom and top of the forms wood wall gages, or spacing pieces, are inserted and the forms wired together, the wires extending around the lateral braces of the forms.

At the intersection of forms the sections are tied together by diagonal wiring. Adjacent forms are nailed together through the side frames, the nails used for this purpose being of such length that when driven to the heads the points protrude a considerable distance on the opposite side. This method of nailing affords great holding strength and permits driving back the nails without injury to the frames when it is desired to remove the

FIG. 2. TOP TIES AND SIDE BRACING OF FORMS FOR FOUNDATION WALLS

forms. At the top the forms are held by U-bars of hoop steel nailed to the outer sides of the forms on either side, similar to the method of holding the forms at the bottom.

With these sectional or movable forms properly set up, and with sufficient wall gages and wired ties nailed at the intersections and secured with the steel hoop bands at top and bottom, the entire construction is very rigid and only a small amount of bracing on the sides is required to steady the forms against shock. Fig. 1 shows the movable forms 9 ft. high, with the bracing. On the ground is the concrete mixing plant. Fig. 2 shows the forms, and the steel holding bands at the top may be seen plainly. These views indicate the small quantity of side bracing required.

The work of erecting forms and placing concrete begin practically at the same time, and on many foundations of moderate size a skilled crew can erect all forms for the basement walls in a few hours.

Fieldbook Sheet for Azimuth

A fieldbook sheet of data for finding true north, used by H. E. McClintock, Assistant Engineer, Union Pacific R.R., Omaha, Neb., is reproduced herewith. The upper table gives the necessary data for observing on Polaris at elongation. The table of time of upper culmination can be used for any year because the small difference for 10 yr., only about 7", permits of accurate interpolation. Eastern elongation is (within a few minutes of time) 6 hr. before, and Western elongation 6 hr. after upper culmination.

In ordinary field practice observations on Polaris are usually the most convenient, and are the most accurate, for azimuth determinations. When conditions make it impossible to observe on Polaris, however, the sun may have to be used.

The following notes of observations illustrate the use of the sheet:

Notes for determination of the true meridian at Devil's Slide, Utah. Lat. 41° 01'. Used Mountain standard time, which equals mean local time for Long. 105° W.

BY POLARIS AT ELONGATION—MORNING OF APR. 25, 1916

Upper culmination (by interpolation) is 23 hr. 31 min. ..... = 11:31 a.m.
Western time is 3° faster than mean local for Long. 108° ..... 12 min.

Western standard time of upper culmination 11:43 a.m.
Eastern elongation is 6 hr. earlier, or ..... 5:43 a.m.

The star appeared to cease eastward motion at about 5:35 a.m. Turned angle to right of 128° 01' to semaphore disk.

Polar distance (by interpolation) is 1° 08½'

Log sin 1° 08½' = 8.296524
Log sin Lat. 41° 04' = 9.877340

Log sin az. = 8.419184
az. = 1° 30' = N 1° 30' E
128° 01'

129° 31'

180° — 129° 31' = 50° 29' or bearing of S 50° 29' E to sem. disk

BY DIRECT SOLAR OBSERVATION—APR. 27, 1916

1. Observed on lower left limb

Western standard time 7:10 a.m. — SH 17° 38' — Hor. Angle Rt. 42° 51' to sem. disk

2. Observed on upper right limb

Western standard time 7:14 a.m. — SH 18° 53' — Hor. Angle Rt. 41° 39' to sem. disk

3. Average of (1) and (2) to center of sun

Western standard time 7:12 a.m. — SH 18° 15½' — Hor. Angle Rt. 42° 15' to sem. disk

Solution of (1)

Refraction for alt. 18° = 0° 03' Semidiam. = 0° 16'

True alt. = 17° 38' + 0° 16' — 0° 03' = 17° 51'

$$\text{Az. correction} = \frac{0° 16'}{\text{Cos } 17° 51'} = \frac{16'}{0.95186} = 0° 17'$$

Correct az. angle = 42° 51' — 0° 17' = 42° 34'

Western standard time is mean local time for Long. 105° W. or 7 hr. slower than Greenwich time; 7:10 a.m. + 7 hr. = 2:10 p.m. Greenwich time of observation.

Declination at Greenwich, noon of Apr. 27, 1916 ..... = North 13° 49' 05"
Hourly change of 47.8" × 2⅛ hr. ..... = 0° 01' 43"

Correct declination ..... = 13° 50' 48"
say 13° 51'

p = 90° — 17° 51' = 72° 09'
s = 90° — 41° 04' = 48° 56'
z = 90° — 13° 51' = 76° 09'

A = 197° 14' ½A = 98° 37'

½A-p = 98° 37' — 72° 09' = 26° 28'
½A-s = 98° 37' — 48° 56' = 49° 41'

Using first formula:

Log sin 26° 28'.... = 9.649020 Log sin 72° 09'..... = 9.978574
Log sin 49° 41'.... = 9.882229 Log sin 48° 56'..... = 9.877340

19.531249 9.855914
9.855914

2)19.675335

Log sin ½z..... = 9.837668 ½z = 43° 29' z = 86° 58'

Center of sun at time of observation bears N 86° 58' E
Corrected az. angle..... 42° 34'

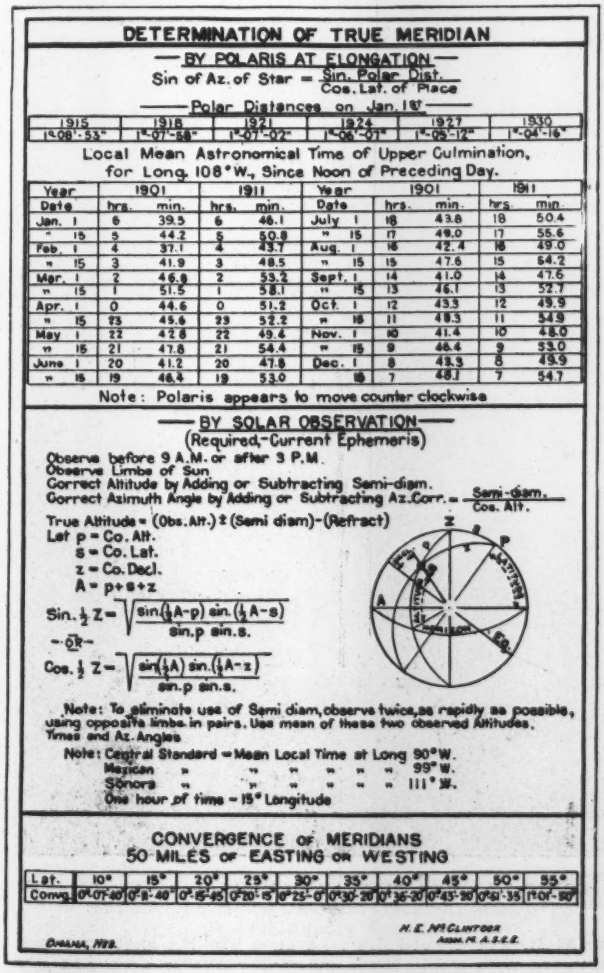
Az. of sem. disk from north..... = 129° 32' = S 50° 28' E

Checks Polaris observation within 0° 01'.

Solution of (2) gives S 50° 29½' E and solution of (3) gives an average of S 50° 28¾' E.

Using S 50° 29' E as correct, an angular tie gives course of track tangent just east of the Devil's Slide as S 48° 30' E.

The table of Convergence of Meridians is useful in checking transit work on long lines. For instance in Lat. 40° N. a line making 25 mi. of Westing (or Easting) would have an error of 18'10" in its corrected course.



DETERMINATION OF TRUE MERIDIAN

— BY POLARIS AT ELONGATION —

Sin of Az. of Star = Sin. Polar Dist. / Cos. Lat. of Place

— Polar Distances on Jan. 1st —

1915	1918	1921	1924	1927	1930
1°08'-53"	1°-07'-58"	1°-07'-02"	1°-06'-07"	1°-05'-12"	1°-04'-16"

Local Mean Astronomical Time of Upper Culmination, for Long. 108° W., Since Noon of Preceding Day.

Year	1901		1911		Year	1901		1911	
Date	hrs.	min.	hrs.	min.	Date	hrs.	min.	hrs.	min.
Jan. 1	6	39.5	6	46.1	July 1	18	43.8	18	50.4
" 15	5	44.2	5	50.8	" 15	17	49.0	17	55.6
Feb. 1	4	37.1	4	43.7	Aug. 1	16	42.4	16	49.0
" 15	3	41.9	3	48.5	" 15	15	47.6	15	54.2
Mar. 1	2	46.8	2	53.2	Sept. 1	14	41.0	14	47.6
" 15	1	51.5	1	58.1	" 15	13	46.1	13	52.7
Apr. 1	0	44.6	0	51.2	Oct. 1	12	43.3	12	49.9
" 15	23	45.6	23	52.2	" 16	11	48.3	11	54.9
May 1	22	42.8	22	49.4	Nov. 1	10	41.4	10	48.0
" 15	21	47.8	21	54.4	" 15	9	46.4	9	53.0
June 1	20	41.2	20	47.8	Dec. 1	8	43.3	8	49.9
" 15	19	46.4	19	53.0	" 16	7	48.1	7	54.7

Note: Polaris appears to move counter clockwise

— BY SOLAR OBSERVATION —
(Required,-Current Ephemeris)

Observe before 9 A.M. or after 3 P.M.
Observe Limbe of Sun
Correct Altitude by Adding or Subtracting Semi-diam.
Correct Azimuth Angle by Adding or Subtracting Az.Corr. = Semi-diam. / Cos. Alt.

True Altitude = (Obs. Alt.) ± (Semi diam)-(Refract)

Let p = Co. Alt.
s = Co. Lat.
z = Co. Decl.
A = p+s+z

$$\text{Sin. } \tfrac{1}{2} Z = \sqrt{\frac{\sin(\tfrac{1}{2}A-p)\sin(\tfrac{1}{2}A-s)}{\sin p \sin s}}$$

$$\text{Cos. } \tfrac{1}{2} Z = \sqrt{\frac{\sin(\tfrac{1}{2}A)\sin(\tfrac{1}{2}A-z)}{\sin p \sin s}}$$

Note: To eliminate use of Semi diam, observe twice, as rapidly as possible, using opposite limbs in pairs. Use mean of these two observed Altitudes, Times and Az. Angles

Note: Central Standard = Mean Local Time at Long 90° W.
Mexican " " " " " 99° W.
Sonora " " " " " 111° W.
One hour of time = 15° Langitude

CONVERGENCE OF MERIDIANS
50 MILES OF EASTING OR WESTING

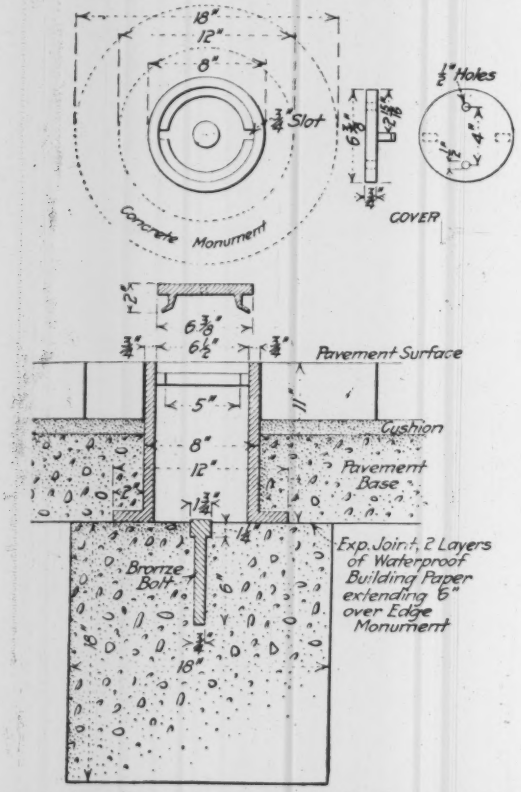
Lat.	10°	15°	20°	25°	30°	35°	40°	45°	50°	55°
Conv.	0°-07'-40"	0°-15'-40"	0°-15'-45"	0°-20'-15"	0°-25'-0"	0°-30'-20"	0°-36'-20"	0°-43'-20"	0°-51'-35"	1°-01'-50"

OMAHA, NEB. H. E. McCLINTOCK Assoc. M. A.S.C.E.

FIELDBOOK SHEET FOR AZIMUTH REDUCED ABOUT ONE-FOURTH

NOTES

City Survey Monument, Houston, Tex.—On account of the peculiar susceptibility to climatic changes of the black waxy soil of the Gulf coast country, a monument separate and distinct from the pavement is needed. In periods of drought cracks develop in the ground as wide as 2 in., and extending several feet deep and hundreds of feet long. When the rains



DETAIL OF STANDARD CITY MONUMENT, HOUSTON, TEX.

come the adjacent soil softens and settles, carrying the pavement with it. On some of the older paved streets in Houston the base has cracked on both sides along lines approximately parallel to and 4 ft. from the curbs extending from one end of the pavement to the other, with the curbs and pavement showing settlement in places as much as 6 in. from the original grade. By constructing a monument independent of the pavement, separating them by two layers of waterproof building paper, the paving base is free to move in response to temperature changes or earth settlements with the least probability of monument being disturbed. Access to the bronze bolt imbedded in the monument marking the alignment or elevation is provided by a casting fixed in the pavement, which is in no way connected with the monument.—J. C. McVea, Office Engineer, Engineering Department, City of Houston, Tex.

Mowing by Machine at an Angle of 45° was accomplished on the canal banks of the Salt River project, according to A. J. Halton, Irrigation Manager, in the "Reclamation Record." The first machine to be purchased required a certain amount of alteration before the cutting bar could be dropped down for cutting at angles of 30 to 45°. This was in 1914. In 1915 the mowing-machine field was surveyed carefully, and three new machines were bought. Experience with these has established the following facts:

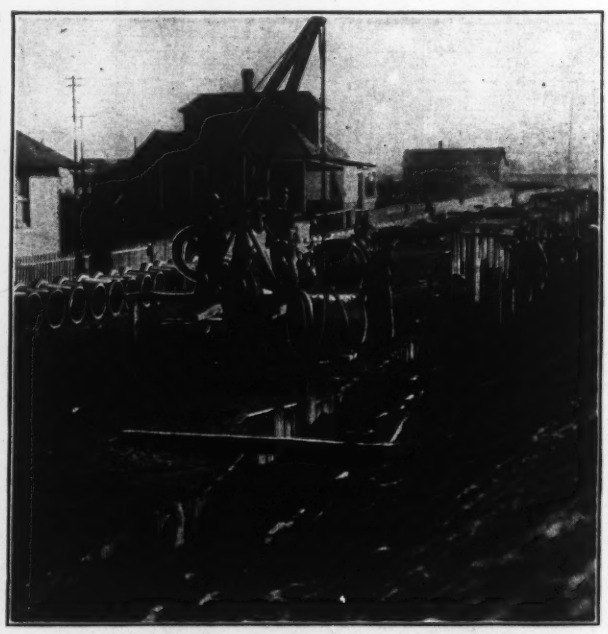
1. It is necessary to have a good teamster skillful in adjusting his machine so it will cut under various and difficult conditions. A quiet, steady team is also essential.
2. It is practicable to use mowing machines on canal banks where the slope does not exceed 45°.
3. Machines cannot be operated on banks where there is trash or heavy brush.
4. Leveling the banks when the ditches are being cleaned greatly increases the efficiency of the machine.
5. Mowing should be done in the early part of the season when the grass and weeds are tender and standing straight. If the work is delayed until late in the season, the weeds fall and make the work more difficult.
6. One mowing machine working under normal conditions will do the work of from 8 to 14 men using scythes, thus effecting a saving of from $10 to $20 per day.
7. With careful driving, machines can be operated in difficult places.

Approximate Expression for Tangent—An algebraic expression claimed to give a very close approximation to the value of the tangent of any angle is presented by George Paaswell, 2726 Creston Ave., New York. The expression is $x\frac{12 - x^2}{12 - 5x^2}$, in which x is in radian measure (360° = 6.2832). It is said to be only 0.2% in error at 45° and 0.7% at 60°. An advantage of the expression, due to its simplicity, is that it can be memorized and used in an emergency when no function tables are handy. It can also be used in algebraic expressions involving transcendental functions where a direct solution in algebraic form is required in order to enable the mathematical work to proceed.

Burning a Wedged Sledge out of a stone crusher by means of an oxyacetylene cutting torch was recently reported at Middletown, N. Y., by the "American Machinist." A workman dropped a 20-lb. sledge into a running crusher in the municipal stone plant. The only alternative to this scheme of removal was to dismantle the machine and platform, which would have required a day or more. The torch freed the crusher in 1½ hr.

Specifying the Progress on Construction Work is a feature of contracts for the Calumet intercepting sewer at Chicago, which work is being built under the direction of the Sanitary District of Chicago. The contractor agrees that the progress shall be such that a certain percentage (in value) of the whole work will be completed by specified dates during the period of the contract. The percentages and dates are listed in the contract. Similar practice is followed on the work of the Little River Drainage District in Missouri. Here the total yardage record for each month necessary for the completion of the work within the contract time has been estimated by the engineer, and when individual contractors fail to make the required monthly progress they are notified of the extent of their deficiency, and are thus warned to speed up the work.

Pipelaying with a Portable Derrick was the method used in installing a sanitary outlet sewer at Butte, Mont. Excavation for this trench proceeded rapidly—the daily maximum being 537½ ft. on Oct. 17, 1914—until the deeper part (shown in the view) was reached. This deep part had to be sheeted, and the groundwater proved troublesome. Here 30



DERRICK LAYING PIPE AT DEEPER SECTION OF SANITARY OUTLET SEWER ON B ST., BUTTE

or 35 ft. was a good day's advance. In the shallow trench a tripod with single block and tackle was used for handling and lowering the pipe. The portable derrick was employed at the deeper trench, where saving of labor was an important item.

Editorials

Mechanical Tunneling in Rock

For many years there have been sanguine inventors who believed they could design a rock-tunneling machine that would cut away rock by mechanical action at a lower cost than the same rock can be excavated in the ordinary manner with explosives. None of these inventors have yet succeeded in placing a commercial machine upon the market. The prospects of the future in this direction must be judged in part with reference to the performances of the past.

The most recent tunneling machine, described in this issue, gives evidence of a high grade of designing ability. This machine is now cutting a short rock tunnel for subway purposes and in doing this it has proved itself able to bore an 8-ft. hole in the hard New York gneiss at the rate of about 9 ft. in 24 hr.—a mechanical feat of no small consequence.

A machine can show advantage over the drilling and blasting method chiefly in two ways: First, in making faster progress or lower cost—the two are likely to go together; second, in making tunneling safer. This second point is of considerable importance and heretofore has been more or less neglected in the consideration of the subject. Fissured rock produces a great amount of overbreakage in tunneling and it is responsible at the same time for most of the dangers which affect the work of driving tunnels in rock.

It frequently happens that a stratum of rock is so traversed by jointing planes and seams as to be in effect a pile of separate stone blocks. Driving a tunnel through such rock by drilling and blasting inevitably means disturbing the material for some distance from the excavation line. Many a tunnel worker has been killed by rock falling as a result of the loosening due to the action of the explosive. On the other hand a boring or trepanning operation in the same rock may easily succeed in producing a smooth hole with undisturbed walls, the separate blocks of the material arching together in mutual support rather than falling away from each other.

In the short experimental tunnel now being driven, for example, the conditions are such as to make it seem quite probable that in proceeding by the usual method of blasting a distinct danger from loose rock would have been created and possibly much timbering would have been required to hold the roof. The hole produced by the tunneling machine, on the contrary, barely shows even superficial indications of the presence of jointing planes and fissures.

But while this consideration may frequently enter as a factor in the choice of tunneling methods, speed and cost will be determinative in the great majority of cases. Time, and development by successive trial, alone can show whether such speeds and such low costs can be attained as to making a tunneling machine a formidable rival of the rock drill and the dynamite cartridge.

It is very interesting to note that the new tunneling machine, like most of its predecessors, operates by chipping action. Percussion tools of some kind seem to have been the chief reliance of tunneling-machine inventors. Far back in the history of the subject, however, trial was made of a rotary cutting machine, and later devices of the same type have been applied to softer material such as hardpan and hard clay. An analogy may be found also in the field of coal-cutting machines, where the material dealt with is something that might be considered a soft rock, and where pure cutting action has been found the successful solution.

The question presents itself, whether rotary cutting action may not in the end prove to be the effective solution of the rock tunneling problem, in hard rock as well as soft rock. The possibility of transmitting unlimited amounts of power to the point of use through electric transmission, and the enormous advance made in recent years in tool steel, create new possibilities in this direction as compared with the situation existing a generation ago.

Rail Troubles Not Yet Ended

If a sure cure has been discovered for the transverse-fissure disease of steel rails, as announced by Dr. P. H. Dudley of the New York Central Railroad in a circular just issued, it will mean a great deal for the safety of railway travel. The strange malady has each year caused hundreds of rail breakages, and wreck upon wreck. Though all the engineering and scientific resources of the railway world have been brought to bear on the problem, the cause and prevention of transverse fissures remained a mystery. Dr. Dudley now says that the cause is an internal crack produced by strains from the gagging press in straightening the rail, and that such cracks occur only in rails which have a nonductile core formed by "delayed transformations." The cure, he says, is reheating the blooms.

The hard-core hypothesis was employed previously by others in a different way. For example, J. E. Howard, of the Interstate Commerce Commission, has practically proved the peening deformation in the rail head to be so great that a nonductile or a weak core, if it exists, is bound to rupture. And such a rupture, of course, is bound to grow outward under repetition of stress. Thus, if we once have the bad core, the case is dangerous, gagging or no gagging. Dr. Dudley's statement that he finds the imprint of the gag on the core in such breaks tends to put a large amount of the blame on the straightening process.

We are left in the dark for the present as to how reheating the blooms will prevent the formation of a bad core; or, for that matter, how a hard core is formed at all, when we know with certainty that the interior of the rail head cools slowly and therefore can anneal fairly well. Mr. Howard and those who follow similar lines of thought seem to blame the bad core on two factors, (1) segregation, and (2) stretching in the rolls, made worse by shrinkage strains. Dr. Dudley, in failing to mention

segregation, practically absolves this ancient bugbear of blame. But in contrast, it is well to recall, another railway pays several dollars per ton for special tests to limit segregation in the head.

At a time when Dr. Dudley's cure for transverse fissures may produce the optimistic belief that rail troubles are now over, along comes the Interstate Commerce Commission with a startling accident report. This deals with a derailment at York Road, on the Western Maryland, last January. A rail broke from serious interior defects—extensive longitudinal seaming in the top-of-ingot rail, due to insufficient cropping. The startling feature is that this break occurred after ten years of service of the rail.

Heretofore the comforting thought has been entertained that defects in a rail will show up soon after the rail is laid in track. We have always felt that after a rail has gone through several years of service it has proved itself to be a good rail. Now we learn that such is not the case. A rail may be dangerous, highly dangerous, and yet not betray its condition until it has nearly reached the end of its natural life.

This rail was smashed up completely for an 11-ft. length; only 56 pieces were recovered. Allowing fully for the hammering effect of the train after a first breach was made in the rail, it yet seems that most of the 11-ft. section must have been in almost an explosive condition—ready to go suddenly upon a slight disturbance. On an optimistic basis one might consider that the condition was a good one: that the rail held until worn out, like the one-hoss shay, and then let go all together. But in reality the condition was bad indeed, and the mystery is why the rail held so long.

The York Road accident thus tells us that the next generation will still have to wrestle with the problem of rail defects. It will inherit from us the troubles being created for it today by present-day lack of skill in steel-making and rail rolling. Even if we were to lay nothing but perfect rails in the track from today on, our railways would not become wholly safe against accidents due to the breaking of defective rails until another decade has elapsed.

New York's Harbor Troubles

With considerable flourish of trumpets it was announced just a year ago that the City of New York had appointed a commission of engineers to investigate and report on the terminal problem in New York harbor, to study the various railroad and steamship terminals and their relation to business centers, in short to prepare a comprehensive scheme for the development of the harbor as would befit what it is fast coming to be—the greatest port in the world. The announcement was no dream of a publicity agent or commercial boomer; it was a carefully considered statement of a branch of the city government. It included not only the names of the eminent engineers selected to serve on the commission but an ambitious budget which involved two years' work and a quarter of a million expenditure.

Probably many of our readers are wondering if this commission is not about ready to issue at least a preliminary report which would give some idea of the proposed solution of the greatest problem before forward-looking American transportation experts—the coördination of New York harbor. Any such expectations are doomed to disappointment; the New York harbor commission has never existed for the simple reason that the Board of Estimate has never provided any money on which it could exist.

Meanwhile the numerous antagonistic interests controlling and using the port go on their dissociated way. The State of New Jersey is complaining to the Interstate Commerce Commission that it is not treated as it should be in the dividing up of the New York harbor business. The railroad, the city, the state and the yellow press have hard work agreeing on the west side freight approaches. The Federal Congress turns down the necessary Hell Gate deepening as a part of the River and Harbor Bill but nearly forgets to eliminate it from its hiding place in the Naval Bill, where it was placed in the hope that a mere half-million would be overlooked in our three-hundred million naval budget.

Taking it by and large there are times when one doubts the infallible wisdom of the founders of the republic and wonders whether a few years—say about one generation—of absolutism wouldn't settle satisfactorily a few of our most annoying problems.

Activated-Sludge Progress

Activated-sludge plants from ocean to ocean and from the lakes to the gulf will be in operation within a few months if present plans are carried out. Milwaukee and Cleveland have had in use for some months working-scale units which are the beginnings of proposed large installations. Baltimore has adapted some of its Imhoff tanks to the activated-sludge process for working-test purposes. The Brooklyn sewer engineers are giving close study to the new process, as is also the staff of the Lawrence Experiment Station. Tests at several different plants in Chicago are being made by the Sanitary District and by packing-house companies. The Illinois State Water Survey experiments at Urbana are being continued in a comprehensive and promising way. A plant is being built if not already in use at Houston, Tex. Plans for a plant in New Jersey have been approved by the State Department of Health. An engineer has recommended Riensch-Wurl screens and activated-sludge tanks for Lima, Ohio. Most recent of all, the authorities of Hermosa Beach, Calif., have decided to build an activated-sludge plant.

In England, progress was checked at an early stage by the war, but the process has been studied at Manchester, Salford, Sheffield and perhaps other places. *The Surveyor* of July 28 states that "Worcester is the only place in England where the most modern system of sewage disposal on a large scale can be seen at work." At Worcester half the sewage of the city was being treated by Jones & Attwood, of Stourbridge, under a provisional contract providing for a year's test, three months only of which had been carried out. About 950,000 U. S. gal. of sewage are being treated daily under this contract, which provides that the effluent shall not contain more than four parts per 100,000 of suspended matter and shall be nonputrefactive. The poorest of five weekly samples showed only 3.8 parts of suspended matter and the next highest one only 1.2 parts. None of the samples gave evidence of putrefaction.

The attention that has been given to this most interesting process of sewage treatment by many leading sewage engineers, chemists and bacteriologists, the number and

widespread distribution of the experimental and practical tests to which the process is being subjected, and the fact that municipalities here and there are putting in trial units and even plants for the whole city may well lead any engineer or city with a sewage-treatment problem in hand to give most careful consideration to the activated-sludge process. Should its highest promises be realized it will fall little if any short of revolutionizing sewage disposal. As yet, there is need for caution. Many details of design and of operation remain to be worked out or, where already worked out, chosen from after further experience.

Although land area required and construction cost are low the cost of operation under varying conditions has not yet been conclusively determined. The process promises to be very flexible and adaptable to meet local conditions but until more working data are accumulated and interpreted those who have had little or no experience with the process will do well to go slowly and thoroughly —following in this the example of those who have had the most experience. It is highly desirable that trial units or plants readily convertible to other processes be installed freely in various parts of the country and that their operation be studied closely by trained observers in order to accumulate and interpret many and varied data as rapidly as is consistent with thorough work.

Dangers in Solid Rock

Revision of the proverbial phrase "firm as bedrock" seems advisable in the light of events showing that the earth's rock strata may fail either below the surface, as in the Scranton subsidences, or at the surface, as in the remarkable Highland Falls case described on another page. Seams, cavities and whatnot create dangers that make rock often more treacherous than running earth. That immense cliffside slab which hung over the bridge at Highland Falls furnished a striking illustration. Happily, ingenious resourcefulness devised a complete protection.

Similar dangers threatened the operation of the Swiss Loetschberg tunnel railway, opened just three years ago. As recently described by one of the engineers of the line, the south approach, from Brigue along the steep mountain sides of the Rhone valley, had a prodigious amount of trouble with loose and sliding rock masses. Masonry underpinning supports solved the difficulty in some locations. Often, however, the safest way was the simplest: swinging the railway line into the inside of the mountain for a short distance by a bypass tunnel. Pinning a loose mass to place, however, has not been done either there or elsewhere. The railway engineers who did the Highland Falls work have a clear title to originality.

No Water Power Legislation in 1916

The adoption of an acceptable water-power bill by the Federal Congress during the present session now appears practically impossible, according to reports from Washington of a complete deadlock between the House and Senate conference committees on measures accepted by their respective bodies. Apparently there is uncompromising opposition on two main ideas dominating the attitudes of the "conservationists" and "utilizationists" who control the respective committees. One is the annual Federal charge for the waterpower rights; the other is special legislative permission for each dam to be built across a navigable stream. With complete disagreement holding up legislation there can be no power development on public lands except under revocable permits of the Secretaries of the Interior and Agriculture; and there can be no dams at all on important waterways.

Strangely enough the major conflict is on what to engineers will seem a very minor matter, in practice if not in principle—the annual Federal charge. Such required payments as have been embodied heretofore in the revocable permits have been trifling sums intended to cover the administrative cost of supervision. It is probable that they would remain so under Acts of Congress rather than run the risk of unconstitutionality as a revenue producer. They might perhaps be arranged in a sliding scale aimed to penalize high rates to consumers and unduly delayed development—witness the well remembered agreement between the Secretaries of the Interior and Agriculture and the International Power Co., for the development of 112,000 hp. on the Pend d'Oreille River in Washington, wherein the minimum tax was 5c. per horsepower-year and the possible average from $1 to $5.

Such a deadlock as now exists in Congress is lamentable, unstatesmanlike, unpatriotic. Failure to lay down some sort of a program that will seem to induce capital to enter the hydro-electric industry is bad; but to the engineer even worse is the failure to prescribe some definite terms under which dams may be built so that the feasibility of the many pending projects can be determined or dismissed.

When a Water-Works Surplus Changes to a Deficit

Under ordinary methods of municipal accounting and financing as a city pays off its water-works bonds the profits increase year by year until important extensions of the works require a large new bond issue. Thereupon the surplus gives place to a deficit. Such a deficit faces New York City within a year or two on account of the immense outlay for the Catskill water-supply, the first unit of which is nearing completion. The case, together with comments on meeting the deficit, is interestingly put by William Williams, Commissioner of Water-Supply of New York City, in his report for the year 1915, just published:

The estimated value of the city's present water-supply system, exclusive of the new Catskill system, was at the end of 1915 $227,000,000, against which there remained outstanding at that date a debt of only $91,000,000. The interest and sinking fund charges against this debt during 1915 amounted to $4,770,000, while the cost of operating and maintaining this system, including taxes, was $4,541,606. Since the revenues for 1915 exceeded $13,000,000, the income account for that year showed a surplus of nearly $4,000,000. Thus the city appeared to be in a very comfortable position as regards the financial side of its water-supply business.

What is commonly known as the Catskill system is being constructed out of corporate stock (bonds). Down to Dec. 31, 1915, $129,175,000 of corporate stock had been issued for this purpose. Further issues will be required before this system is completed, but the interest and sinking-fund charges on the amount issued down to Dec. 31, 1915, reach $7,500,000. As soon as the operation and maintenance of the new system come under this department, as will probably be the case by Apr. 1, 1917, the department's income account from its water-supply business will cease to show a surplus of any kind, and, on the contrary, will show an annual deficit ranging between $2,500,000 and $3,000,000, unless a means can be found materially to increase the water revenues. When this happens there will be a rude awakening for many who have come

to regard New York's water-supply as a source of financial profit. At that time fewer people will be heard to demand that water shall be as "free as air," and more will come to view it as a most valuable commodity, to be used with care and not wasted, as now so much of it is. At that time, also, the department's efforts to bring about the abolishment of the unintelligent and discriminatory frontage rate system and the installation of meters on all services, so that all water used shall be paid for at a proper rate and that none shall escape (as many now do escape) their fair share of its cost, will be more generally appreciated than they are at the present time.

This situation suggests many points, only a few of which can be discussed here. First, it may be noted that New York, like most other cities, furnishes a large amount of water to city departments and to public or semipublic institutions for which the water department gets no credit. According to Mr. Williams this free water in New York, at current local meter rates, amounts to $2,000,000 a year. This sum might very properly be added to the present yearly surplus. It should just as properly be deducted from the deficit Mr. Williams foresees. Second, if the water consumers of New York have been paying not only for the water they have used but also for that used by city departments and various institutions they have either been overcharged or else there would have been accumulated by this a large surplus fund to offset a part of the Catskill water debt. Third, the Catskill supply, with its higher pressure and better quality, is going to be worth more to the consumers than the water they are now getting. Fourth, a portion of the Catskill supply is to meet increase in consumption due to growth of population and industry, which of course should foot the bills; but another portion of the supply and corresponding outlay is due to provision for the future, which makes a sudden and high jump in capital charges with no immediate effect on water revenue.

A large part of this jump, in a commercial utility's financing, would be made a burden on the future consumers for whose benefit the excess capacity has been secured. This would be done in some cases by capitalizing the deficit, as a cost of developing the business, with provision for later amortization. In the case of a municipality the same result might be secured by issuing bonds to cover that part of the deficit properly attributed to the cost of excess capacity.

Finally, coming back to the concluding part of the quotation from Mr. Williams' report, the present system of charging for water in New York City is not only antiquated but was never justifiable and is notoriously unjust. The frontage charge alone goes far toward explaining why increased water consumption does not mean a proportionate increase in revenue.

Letters to the Editor

Masonry Arch Makes a Cheap Highway Bridge

Sir—Apropos of the editorial on "Concrete and Stone Masonry for Bridges," in your issue of July 20, I send a photograph of a simple highway bridge built in this town a few months ago.

Here, cement and broken stone are obtainable at a little more than average prices, because of freight rates. In this case there was plenty of stone available, and local masons not particularly skilled in laying bridge masonry. After getting a rather high bid from a contractor, we decided to do the work by force account, utilizing home talent and spending the money chiefly in the town.

The span is 28 ft., rise of arch 5½ ft., springing lines 5 ft. above the base. Foundations are of concrete rubble, built behind shallow wooden sheetpiles on gravel bed. The abutments, below the spring lines, are 6 ft. thick. Strong cement mortar was liberally used throughout the work. The arch between stone facing is made of concrete, reinforced with expanded metal. It is intended to carry any possible load up to a locomotive.

The concrete is probably more expensive for us than stone. Indeed we built a second bridge, using the same centers across the same brook, all of stone, the cost of the two bridges being less than $500 more than the contract bid for one.

The photograph was taken before the clean-up of the ground. While the coursing of the masonry may seem a little crude, it must be noted that not one passer-by in ten thousand ever sees more than the parapets on their inside.

The selectmen were persuaded to incur a little greater first cost in order to have a bridge needing no charge for maintenance, and one equal to any traction engine or other future load. Once for all they were rid of worry about capacity of bridge, repainting (had they chosen a plate-girder), replanking floor or renewing pavement, etc.

CHEAP AND ATTRACTIVE MASONRY HIGHWAY BRIDGE

ROBERT FLETCHER,
Director, Thayer School of Civil Engineering,
Dartmouth College.

Hanover, N. H., Aug. 11, 1916.

Creeping and Heaving of Concrete Road Slabs

Sir—In the editorial on the creeping and heaving of concrete road slabs, June 22, 1916, p. 1193, you attach great importance to proper construction of the subgrade—something beneath the concrete and not an integral part thereof—which suggests that it may not be irrelevant to refer to the success achieved by putting something on top of concrete—for example, an asphaltic concrete wearing surface.

If we separate highways in which concrete has been used into two classes, those consisting entirely of concrete and those protected by a wearing surface, we find that in the latter type concrete has given little trouble, whereas unsatisfactory results are far more common in the case of roads constructed entirely of concrete.

According to Frank W. Sampson's communication, published in *Engineering News*, July 20, p. 134, the creeping and heaving of concrete occur when the slab becomes saturated, followed by drying. In discussing the subject recently with Henry H. Quimby, Chief Engineer of the Department of City Transit, Philadelphia, I found that his opinion coincides with the view expressed by Mr. Sampson. Mr. Quimby also believes that a water-shedding wearing surface would remedy the difficulty and provide further desirable protection against sudden and extreme changes in temperature. I find that others who have made concrete the subject of careful study for years share Mr. Quimby's convictions as to the efficacy of a wearing surface.

The proposition involves of course the matter of cost. I find on this point that Northeast Boulevard, Philadelphia; streets in Newton, Kan.; a section of the Belair road, just out of Baltimore; and a road in Monroe County, Mich., all of them having a 4- or 5-in. concrete base and a 2-in. asphaltic concrete wearing surface, cost less per square yard than the present price of cement concrete roads.

To the best of my knowledge the concrete on these thoroughfares neither creeps nor heaves. Do we require on country roads a better type of pavement than Philadelphia put down on Northeast Boulevard for about $1.25 per sq.yd.? Has enthusiasm on the subject of good roads caused the pendulum of high cost to swing from the metropolis to the country?

E. A. TREGO,
Barber Asphalt Paving Co.

Philadelphia, Aug. 8, 1916.

Curves for Irrigation-Ditch Velocity and Discharge

Sir—In the article by L. R. Douglass, on "Curves for Irrigation-Ditch Velocity and Discharge," in *Engineering News* for July 13, 1916, p. 72, the author has evidently undertaken a very difficult task if he is attempting to determine accurately the "maximum velocities and discharges" for an old ditch from computations based upon a multiplicity of cross-sections. Where a canal is in operation, the simplest and most accurate way to find the maximum capacity is to turn into it as much water as is thought safe and take a few current-meter measurements. However, conditions were doubtless such as to render the method of procedure adopted necessary and further discussion will be limited to the diagrams proposed by Mr. Douglass for the solution of the hydraulic problems involved.

The statement made by the author that his diagrams are designed to eliminate the "long and tedious" computations by Kutter's formula indicates a lack of familiarity with published data upon the subject. There are probably more curves in existence for the solution of Kutter's formula than for any other one formula. In "Working Data for Irrigation Engineers," by Moritz, a series of curves is given for the complete solution of such problems as the one presented. Having the dimensions of the sections, the slope and Kutter's n given, the area, the hydraulic radius, the velocity and the discharge are determined from Moritz's diagrams without computation. The curves cover a wide range of canal sections and give velocities for all the commonly used values of n. As no diagram is given for $n = 0.027$, it is necessary to interpolate between $n = 0.025$ and $n = 0.030$. A value of $n = 0.027$ is rarely encountered, and ordinarily the interpolation method serves every purpose; but in cases like the one cited, where a great number of computations involving an unusual value of n are to be made, special diagrams may be required. It is desired to call attention to some points in connection with the curves submitted by Mr. Douglass.

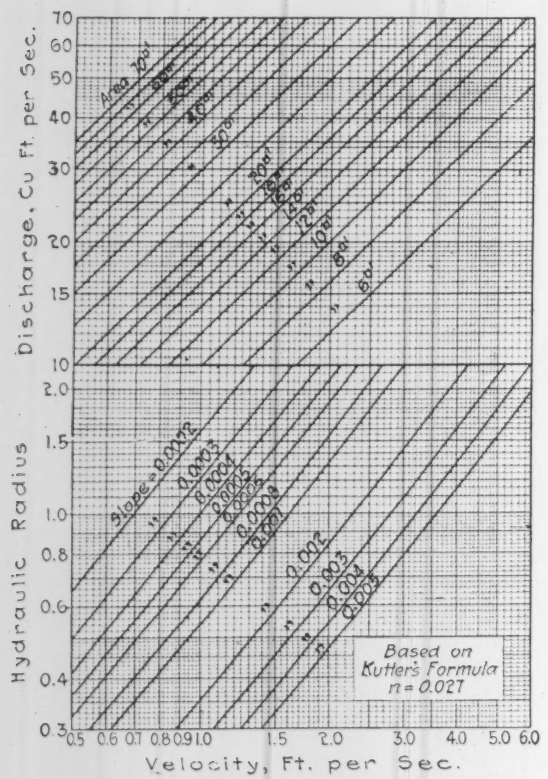
The diagram, Fig. 1 (p. 72), is less accurate and less convenient than the slide rule and is of little value. The upper and right-hand scales in Fig. 2 (p. 72) likewise perform a simple operation that ordinarily could be performed more easily on the slide rule, but these scales may be used to advantage in connection with the other scales to obtain both the velocity and the discharge from a single entry. This may be done by entering the diagram with the slope and the hydraulic radius, as directed, to find the velocity, thence passing around the velocity curve to intersect the horizontal through the area, and thence vertically to the discharge. This point was not brought out clearly in the article.

It should also be noted that there is a small theoretical error in the construction of Fig. 2. In order for the set of velocity curves to exactly fit both of the two sets of coördinates it would be necessary for the velocity to vary as some simple power of the slope. This is not strictly true, but it is probable that within the limits of the diagram the error is negligible. The diagram has, however, one serious defect. The velocity curves for almost their entire length are nearly parallel to one or the other of the coördinate systems. This makes it difficult to read the diagram accurately. To eliminate this difficulty the curves should be plotted on logarithmic cross-section paper. In this case the velocity curves would be nearly straight lines and would intersect the coördinate lines at practically a constant angle. Also, the curves being approximately straight, fewer computed points are necessary for plotting. Logarithmic curves give a greater range of values on a small area with an equal percentage of accuracy over the whole diagram.

Granting that the number of computations to be made using $n = 0.027$ is sufficient to make the work of interpolating between existing curves or tables greater than the work of constructing a special diagram, the arrangement shown in the accompanying diagram is suggested. The two sets of curves are separated for clearness and to remove all doubt as to accuracy. It will be noticed that the diagrams cover the same range as is covered by the

original curves and that the angle of intersection is good throughout.

Only a sufficient number of lines to illustrate the principle are shown in the figure; other lines can be added at will. The lines in the lower diagram are very nearly straight and are plotted from velocities taken from "Hydraulic and Excavation Tables," published by the United States Reclamation Service. The area lines in



CURVES FOR FINDING VELOCITY AND DISCHARGE OF OPEN CHANNELS

the upper diagram are straight lines inclined 45°; and since area is equal to discharge when the velocity is 1, they are readily drawn. The straight lines and favorable intersections do not result from the rearrangement, but are due to the logarithmic cross-sections. If the velocities are plotted as in the original diagram, they will be very similar to the slope lines shown here.

JULIAN HINDS,
Asst. Engr., United States Reclamation Service.
Denver, Colo., July 26, 1916.

Comment on Civil Engineers' Flood-Control Report

Sir—Since the submission of the "Progress Report on Floods and Flood Control" by the committee of fifteen appointed by the American Society of Civil Engineers, there has been a great deal of commenting and criticism on the report by sundry engineers. The criticisms that have been offered, of an adverse sort, have been little more than verbal quibbling over matters of phraseology; and the discussions generally have related to relatively inconsequential features of the major problem.

But critics and commentators have all missed the kernel of the matter, which is, that it is not possible to formulate a report that shall meet the implied demands of these critics, and cover in a comprehensive way what is embodied in the title of the report, as applicable to the entire territory of the United States.

The difficulties in the way of such a comprehensive report as the critics by implication demand, cropped out very soon after the committee began its labors. Colonel Townsend, chairman of the committee, as an initial step, and after consulting the members of the committee, divided the body into sub-committees by geographical distribution, and requested partial reports from the sub-committees to serve as a basis of a general report. In response to that request came but one report from a sub-committee as such, that on the Ohio River. Several other reports were submitted by individual members.

But each report received was an elaborate treatment of *one individual river*, with no reference to any other river; and in the nature of the case this is what must have been expected.

Glancing over a map of the United States and noting all of its drainage systems, it is seen that there is a large number of rivers of sufficient size and importance to engage the attention of engineers; and every one of these rivers is strictly individualized, requiring a special treatment to fit its individual characteristics, upon a plan and by methods that are not applicable to other rivers; and on some of the larger rivers different portions of the same stream are strongly individualized, differing radically one portion from another, and requiring radically different treatment.

A general report to cover all of these rivers with entire justice must resolve itself into an agglomeration of reports on all the individual streams. Any report that shall embody basic principles that are applicable to all of the rivers in this heterogeneous assemblage of highly differentiated units, must perforce be a meager covering, consisting of the few and obvious principles that are common to all.

An analogous case would be to demand of a committee of the medical faculty of the whole country a general report that should cover the treatment of all the diseases in the category of therapeutics.

T. G. DABNEY,
Clarksdale, Miss. Member of Committee.

Electrolytic Sewage Tests at Elmhurst and Decatur

Sir—Those interested in sewage-treating processes have doubtless read the statement relative to the electrochemical treating plant at Elmhurst, Borough of Queens, New York City, in your issue of July 13.

The writer visited the Elmhurst plant a year ago, and while he found details to be criticized from an engineering standpoint, the results that were being obtained indicated great possibilities and the process showed merits worthy of very careful consideration.

The equipment had been installed in a plant designed for another process and the whole installation was not what the process required for best results. Prices paid for electricity and lime were excessive, and the plant was treating 750,000 gal. per day at a labor cost sufficient to handle several million gallons.

The following results, however, were evident: The treatment was made quickly and without odor or nuisance. The effluent was bright and clear and created no objectionable effects below the plant. The sludge was in a condition to be pressed while fresh, and the cakes were easily handled and remained practically inodorous.

It has been my privilege within the past two months to design and see put into operation a demonstration plant wherein this process is used under more favorable conditions. The plant is located in the City of Decatur, Ill., and has a capacity for handling 1,000,000 gal. of sewage per 24 hr.

The sewage is drawn from the main sewer of the city, a 7-ft. brick sewer that carries some three to four million gallons of very strong city sewage, including the trade waste of a large slaughter and packing house, a gas plant, a starch manufacturing plant and a zinc-plating works. The starch works alone furnishes practically a million gallons of waste that is particularly difficult to treat, as it is strongly impregnated with sulphurous acid and is not susceptible to biological treatment.

The plant is located within six blocks of the business center and is housed in a neat frame structure 18x70 ft. in plan. In this is installed a motor-driven pump to lift the sewage some 18 ft. into a feed box, a 1,000,000-gal. electrolytic treating machine with its motors and special generating sets, switch boards and other equipment; a motor-driven sludge pump, and (for demonstration purposes only) a crude-oil engine for generating electricity, thereby giving a complete self-contained plant.

Outside the building is a settling tank 18x60 ft. in the bottom of which are placed four conical hopper-bottom pits. The tank is lined with lumber and has a total depth of 16 ft. to the bottom of the pits, the hoppers extending from the bottom to within 6 ft. of the flow line. The tank is not covered, and sludge pipes are provided extending to the bottom of each hopper through which sludge is constantly drawn off by the pump, and forced through a Johnson filter press located beneath a shed outside the building.

Over a portion of the building is a second story in which are the lime solution and storage tanks. Outside and independent of the building are two small sheds, one for storage of lime, the other a laboratory for the use of the chemists, the entire plant, treating 1,000,000 gal. of sewage daily, occupying about half an acre of land.

The sewage is drawn from the main sewer through an opening in the bottom so that the plant gets its full share of the heavier suspended particles that are carried along the bottom of the sewer. It passes into a pump pit in which a screen is placed to remove all coarse trash that will not pass through ½-in. circular openings. A 5-in. centrifugal pump lifts the sewage from this pit into a feed box containing a weir and overflow, arranged to control the quantity of sewage fed to the machine. Owing to the odors produced by the starch wastes, this box is sealed tightly in a separate room with only a glass gage showing in the operating room to indicate the height of flow over the weir. By-pass pipes are provided, one in the feed box, one in the entrance trough where the treated sewage enters the settling tank, and a second at the outlet end of the tank. These all connect with a tile drain that carries the water back to the main sewer and discharges it below the inlet. A surface outlet is provided through which the treated effluent may enter an open ditch, and after traveling some 300 ft. down a valley enters a branch sewer through which it flows back to the main sewer. The character of the sewage treated compares favorably in appearance to that coming from the Chicago stockyards and is more concentrated than the sewage used in the experiments at Milwaukee.

The plant was operated for some two weeks in order to make adjustments and such changes as were found necessary to meet the requirements of the sewage, and was then turned over to chemists representing the city of Decatur and the Electrolytic Sanitation Co., of Chicago (which installed the plant), for a determination of the results.

To all appearances the plant is doing all that was expected of it. Odors are destroyed, the sludge settles readily into the hoppers and presses into a cake that is easily unloaded from the press, and the effluent on leaving the tank is bright, clear and practically odorless. The cost has not yet been fully determined, but indications are that it will compare most favorably with any of the other known processes, especially when the amount and character of waste from the starch works are taken into consideration.

W. S. Shields,
Consulting Engineer.

1201 Hartford Building, Chicago, Aug. 7, 1916.

Not So Dry in Payta

Sir—In looking up some information on rainfall, I found in your issue of Apr. 19, 1900, p. 249, a note on the "Rainless Region" near Payta, Peru.

I spent three years, 1912-14, in Peru—about two-thirds of the time near Payta. Less than 100 mi. southeast of Payta, I recorded over 5 cm. in one night, and on Mar. 1, 1915, I encountered the prettiest little rainstorm one could wish for at Payta and about 100 km. west of there. It put out of commission the only irrigation canal in that vicinity for four months, on account of the caving in of the banks.

The note in question also stated that the average time between showers is seven years. You would be stretching the truth if you said that the present average duration between showers is seven months. During the time I was there I never saw more than three months between showers.

Several inches of rain falls there during the year, and especially during the rainy season of December, January and February.

Walter S. Wheeler.

30 East 42nd St., New York City, Aug. 9, 1916.

[If 16 years ago we gave Payta an unwarranted reputation for rainlessness and if we are thereby in any measure responsible for the failure of the irrigation canal, we are sorry.—Editor.]

NOTES AND QUERIES

The Surveys for the Proposed Colorado River Dam, described on p. 306 of "Engineering News" of last week, are referred to a base line 780 ft. long instead of a 200-ft. base, as noted in the article. The mouth of Diamond Cañon is about 800 ft. wide.

In a News Note on p. 766 of "Engineering News," Apr. 20, 1916, it was stated that W. C. Abbott, of the Smithsonian Institution, was coöperating with a weather observer in Houston, Tex., in research work which would make possible long-range weather forecasts. The Institution informs us that no such coöperation exists.

Structural Design of a Peaked Reinforced-Concrete Roof

By J. Norman Jensen*

In the construction of the New Bancroft Hotel in Saginaw, Mich., reinforced concrete was used throughout. There were no unusual structural features in the main portion of the building, but the banquet hall, about 40x70 ft. in plan and one story high, presented a somewhat novel type of roof construction.

*2504 North Sawyer Ave., Chicago, Ill.

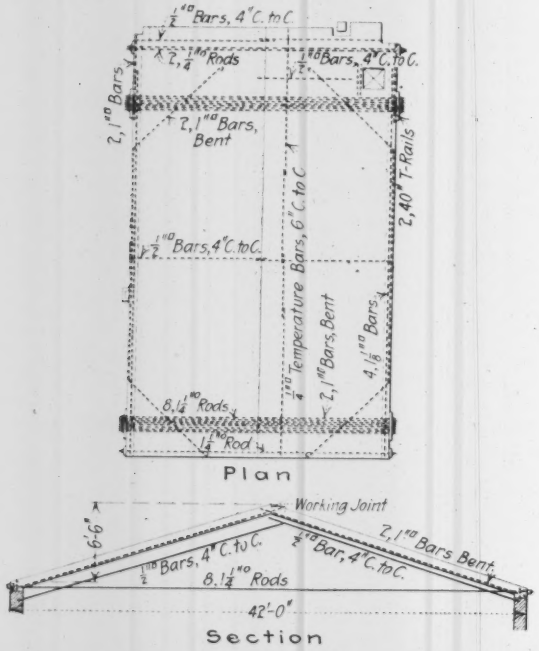


FIG. 1. DETAILS OF PEAKED CONCRETE ROOF AT HOTEL AT SAGINAW, MICH.

The architectural requirements of this portion of the building were that a peaked roof should be used, an arched ceiling should extend the greater portion of the hall, and that any girders or tie-rods should only be used in the extreme ends of the ceiling where they could be concealed. It was also desired that this roof should be of the same type of construction as the rest of the building.

As the span, 40 ft., was rather excessive for an economical roof slab, and as any level slab would necessitate increasing the height of the exterior walls owing to the arched ceiling, it was decided to design a peaked-roof slab to span from wall to wall. The method of design and computations for this roof are given below.

To find the load per lineal foot of slab, on a 20-ft. span (see Fig. 2):

		Lb.
Live	25 × 21 =	525
9-in. slab	112 × 21 =	2,352
Roofing	3 × 21 =	63
Ceiling	10 × 20 =	200
Total		3,140

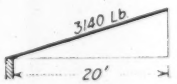


FIG. 2

Using the notation for reinforced concrete as adopted by the Joint Committee, $M = \frac{1}{8} \times 3{,}140 \times 20 \times 12 = 94{,}200$. With $f_c = 700$, $f_s = 18{,}000$, $K = 113$, $p = 0.0072$, $d = \sqrt{\frac{94{,}200}{12 \times 113}} = 8.35$ in. 9-in. slab used. $A = 12 \times 8.35 \times 0.0072 = 0.72$ sq.in. per ft. of width. ½-in. square bars, 4 in. c. to c. were used. Temperature bars were placed in the top portion of the slab as shown on the roof plan and section in Fig. 1. In order to insure that each half of the roof should act as independent slabs, a construction joint was left in the peak of the roof.

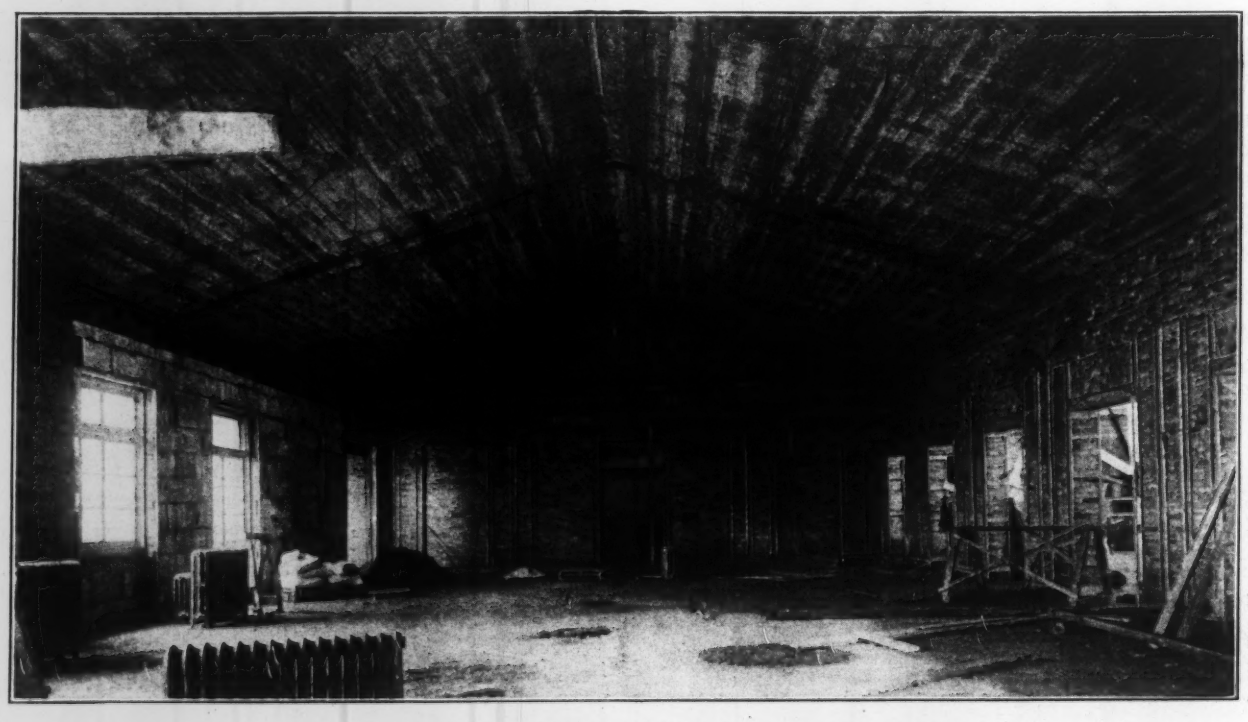


FIG. 3. VIEW OF UNDER SIDE OF ROOF BEFORE ARCHED CEILING WAS PLACED

Having designed the slab, the next step was to design the roof considering it as a simple truss, in which the top chord was the concrete slab, and the bottom chord the tie-rods at each end of the hall. On this basis the thrust per lineal foot of slab $= \frac{3{,}140 \times 22}{2 \times 6.7} = 5{,}150$ lb., and tension per lineal foot of slab $= \frac{3{,}140 \times 21}{2 \times 6.7} =$ 4,920 lb.

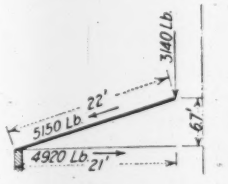


FIG. 4

Since the walls were not designed to take the thrust from the roof slab, it was necessary to consider the roof slab as a lateral girder, of a depth equal to the distance from the peak of the roof to the wall, of a width equal to the total thickness of the slab, and of a span equal to h, the distance between the two sets of main tie-rods. On this assumption, total load of lateral girder $= 54 \times 5{,}150 = 278{,}000$ lb. $M = \frac{1}{8} \times 278{,}000 \times 54 \times 12 = 22{,}500{,}000$. Since $b = 9$ in. and $d = 21$ ft., solving for K. $K = \frac{22{,}500{,}000}{9(21 \times 12)^2} = 40$. $f_c = 350$, $f_s = 16{,}000$, $p = 0.0027$. $A = 9 \times 21 \times 12 \times 0.0027 = 6.15$ sq.in. Four 1⅛-in. and two 1-in. square bars were supplied in each set of tie-rods.

$f_v = \frac{278{,}000}{2 \times 9 \times 21 \times 12 \times 0.91} = 68$ lb. As the unit shear was a little higher than is allowable on plain concrete, the two 1-in. square bars were bent up as shown on the plan to provide additional shear reinforcement.

Each set of main tie-rods carries about 32 lin.ft. of roof. Total tension in rods $= 32 \times 4{,}920 = 157{,}000$ lb. Area required $= \frac{157{,}000}{16{,}000} = 9.82$ sq.in. Eight 1¼-in. round rods, with upset ends and turnbuckles were provided.

To bring the load from the top chord into the tie-rods, bearing plates were necessary. Using 400 lb. as the allowable unit bearing stress on the 1:2:4 concrete used, bearing area required $= \frac{157{,}000}{400} = 392$ sq.in. The two 40-lb. T-rails shown gave a larger bearing area than required.

The analysis will not be quite complete without an investigation as to the combined compression due to the direct compression of the slab acting as a top chord, and also the compression due to flexure in its action as an ordinary slab.

		Lb.
Direct compression	$= \frac{5{,}150}{9 \times 12} =$	48
Flexural compression		700
Total		748

This total value is not excessive for the concrete used.

A view of the completed roof is shown in Fig. 3. The tie-rods at one end of the building can be seen. These rods had not been fireproofed at the time of the taking of this picture, but were afterward wrapped with wire cloth and plastered with cement mortar. The writer is indebted to the architects, Schmidt, Garden & Martin, of Chicago, for the information concerning this unusual roof.

New York State Barge Canal in Partial Operation

The New York State Barge Canal has been under construction for over ten years. It extends from the Hudson at Waterford to Tonawanda on the Niagara River with side canals from Three Rivers up to Lake Ontario at Oswego, from Waterford to Lake Champlain and from Cayuga and Seneca Lakes to the main canal near Montezuma. Progress toward completion has been considerably slower than was at first anticipated but apparently the end is now in sight. The *Barge Canal Bulletin* gives the following statement of the sections of the canal which are under operation during this summer.

From the eastern terminus at Waterford the new Erie is now open as far as Jacksonburg. Throughout this extent the new channel lies chiefly in the Mohawk River. The short portions of land-line in this stretch have no towing-path and of course the river sections have none. Accordingly all towage must be by tugs, unless the canal boat has its own power. Through this new section the State does the towing, an appropriation having been made therefor by the last Legislature. At Jacksonburg the old canal is entered and must be followed until Newark is reached. Here again the new channel is in use, extending as far west as Pittsford.

With the exception of the new line to the south of Rochester, most of the Barge Canal west of Newark consists of a widening and deepening of the old channel. The Barge Canal scheme does not contemplate the use of animal power for towage and therefore no towing paths are provided. However, along most of this enlarged channel in the western part of the state the old towing path has been retained, and raised or lined if necessary. This has been done for the purpose of maintaining navigation during the period of construction, in accordance with the mandate in the Barge Canal Law. The section between Newark and Wayneport is an exception to this rule and here tugs are necessary. The state is providing towage here this year, the same as in the Mohawk River channel. From Wayneport west, however, the boatmen must provide their own power.

The old line through Rochester is employed this year, since the new alignment, which circles the city on the south, cannot be utilized until certain deferred work on it has been completed and also until there has been built in the Genesee River the dam which will provide both the pool on which boats may cross the river and also the spur and harbor, by means of which the city may be reached from the new canal. From the point where the old and new lines meet, near Greece, west to Tonawanda, the new canal, which is almost entirely the enlarged old canal, is open to traffic. A towing-path borders this channel and animal power may be used. A quarter-mile stretch from Tonawanda to the Niagara River is still to be built to complete the Barge Canal, but this year boats will follow the old waterway to Buffalo.

The new Champlain and Oswego canals are open this year throughout their entire lengths. Each branch is largely a river canalization, on which tugs or power-boats are needed. The State is not providing the towage here. The Oswego canal as it exists this year includes two sections which eventually will not belong to it. One is the canalized Seneca River from Three River Point to Mud Lock, near the foot of Onondaga Lake, which is really a part of the Erie Canal, and the other is the old Oswego Canal from Mud Lock to the junction with the old Erie in Syracuse.

Only the eastern portion of the Cayuga and Seneca Canal was opened at first this year. The old channel from the Erie junction into Cayuga Lake was used. In the section opened later the route extends through the new canal from Cayuga Lake to Seneca Lake, but the full depth of 12 ft. has not yet been provided at all places. From the outlet, at the northeastern extremity of Seneca Lake, to Geneva, the course follows the old canal, which extends some two miles and a half around the foot of the lake.

The sections of Barge Canal which are being used this year are not the only portions which are ready for operation. There are certain parts (and they embrace most of the sections not already in use) which can be thrown open as soon as a few uncompleted pieces of work are finished. The new channel between Sylvan Beach, at the eastern end of Oneida Lake, and Newark is thus ready for operation, with the exception of three short stretches. One of these lies between Clyde and Lyons, another is situated near Montezuma, and the third extends from Three River Point to Onondaga Lake and is now in use, serving as a part of the Oswego Canal.

Sinking a Shaft by Dredging Through Springs and Gas

Clay, quicksand and gas-carrying strata gave much trouble in sinking a shaft for the new Central Ave. water-main tunnel under the Cuyahoga River in Cleveland recently. The sinking was effectively stopped at a little over 100 ft. depth below surface. However, since a satisfactory level for the tunnel was found above this point, the tunnel work was able to proceed and has just been completed with entire success.

The difficulties were met in sinking the east shaft of the tunnel, which is located on Stone's Levee on the line of Central Ave. The west shaft, on the opposite side of the river, was sunk later and did not go to such depth, thereby avoiding similar trouble in all probability.

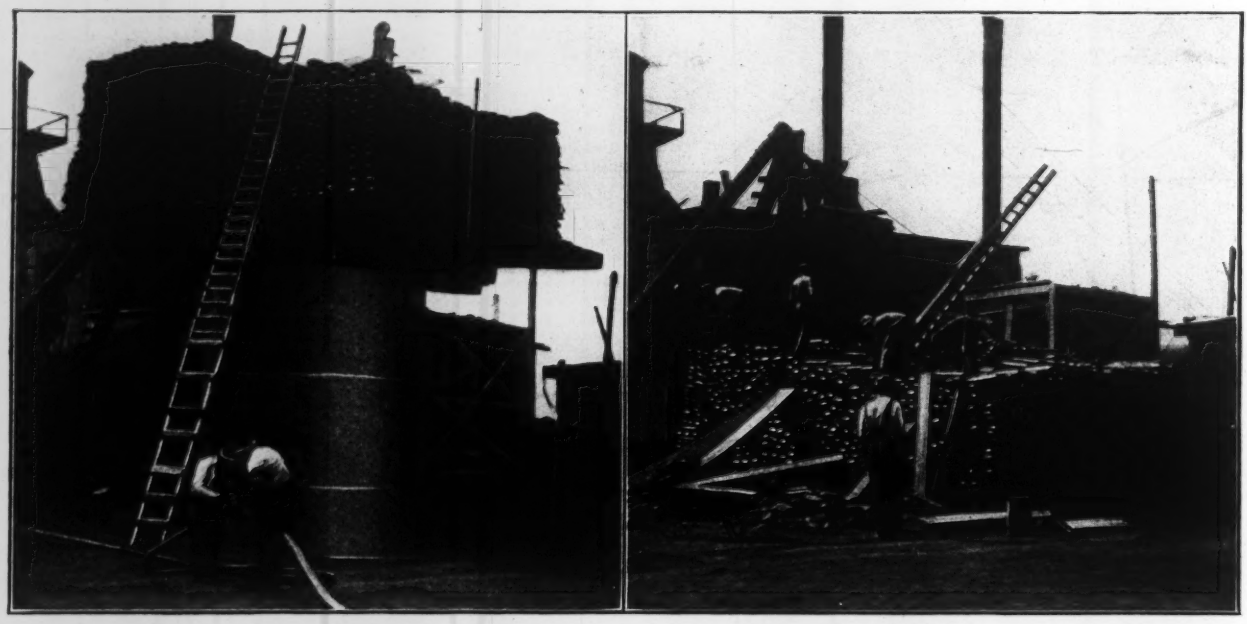
The shafts were originally designed to go down to 121 ft. below river level, this being just below the designed level of the tunnel. The east shaft was started inside an 18-ft. square pit within sheeting driven 16 ft. deep. The steel shell, 13 ft. in diameter, was built together in 16-ft. lengths that were erected successively as the shaft was sunk. At the start the cutting-edge section was turned upside down and concreted with beveled inner edge; then turned over and the next section connected on, and sinking started. Each section as it was joined on was lined with the 12-in. concrete lining.

Digging was done with an Owen bucket having specially wide teeth. The material in the upper strata was dry, hard clay for the most part, but lower down numerous sand seams were encountered. Only one shift was worked, a day shift, and probably in consequence of this there was frequent trouble from the cylinder hanging the next day, due to the clay squeezing and gripping the shell.

About 40 ft. down, a 16-in. log was encountered lying straight across the cutting edge. There was no way of handling this from above. A diver cut it through with an air drill and put a chain around the log, by which it was hoisted out.

At 76-ft. depth quicksand and gas were struck, and doubts at once arose as to whether the sinking should be continued or the shaft bottom sealed. There was several days' delay in trying to reach a conclusion as to the best procedure, and then the decision was to go on with the sinking. Excavating continued, but it developed that the shaft had frozen in place during the stoppage. A week's work did not start it. Meantime pig-iron loading was being put on, which finally reached the amount of 300 tons. Then the shaft suddenly dropped 17 ft. in a few seconds, forcing a large amount of material at the bottom up into the shaft. (See accompanying views.) However, the drop did no damage, but on the contrary cut off the gas and the quicksand.

When 100-ft. depth was reached, there was no more steel shell available, and it was decided to go down 20 or 30 ft. farther by excavating and underpinning. Sheeting was driven, inclined outward over a ring of timbers. This sheeting was a 6-ft. length. A similar length was driven below it; and below this a third, as the digging by pick and shovel went down.



SUDDEN DROP OF SHAFT UNDER 300 TONS PIG-IRON LOADING

The two views were taken a few minutes apart; the drop took only a few seconds, however

At 113 ft. the bottom blew up with a large inrush of natural gas and water, there being apparently a gas-carrying spring of some size. Conditions looked so threatening that the shaft was promptly filled with water by fire hose to stop further disturbance of the bottom.

A pipe was put down from the surface to a depth of 130 ft. to find hard strata. This pipe indicated that reasonably hard ground would be found at 130 ft., which was some feet lower than originally expected. However, on studying the record of the strata penetrated it was decided to raise the elevation of the tunnel about 30 ft. to El. —76. The attempts to carry the shaft down deeper were therefore abandoned, and the shaft was bottomed.

To seal the shaft, 5 ft. of concrete was deposited in the bottom through 100 ft. of water. The concrete for

this course was lowered down through the water in a bottom-opening bucket. On pumping out the shaft the seal concrete was found to be unsatisfactory, gas having formed a bad porous spot in it. To remedy this condition a 3-ft. funnel was placed over the gas pocket and a 4-ft. course of concrete was deposited around the funnel, lowering the concrete through 10 or 15 ft. of water. Thereupon a hole was discovered just outside the funnel, through which gas discharged. This hole was calked with oakum and another 4-ft. depth concreted, making the total depth of concrete about 13 ft.

The water and gas inflow were now under control, except for what came through the funnel and was piped to the surface. Operations then started on the tunnel, leaving the final finishing and grouting of the shaft bottom for a subsequent time.

The west shaft, started later than the other, was somewhat behind and was in the neighborhood of 70 ft. down when the decision was reached in the case of the east shaft to locate the tunnel with flow line at El. —76. For this reason the west shaft was stopped at —75. Throughout its sinking no difficulties were encountered, and the shaft went down without sticking. Within 6 in. of the bottom, however, a strong cold spring broke in. This, as in the case of the east shaft, gave trouble by making the concrete seal porous and leaky, but it was controlled by catching the spring in a funnel and concreting the bottom, and at the completion of the tunnel grouting the bottom under pressure.

The work was done under the direction of C. P. Jaeger, Commissioner of Water, with John W. Dolman in charge of operations. It was carried on by force account. Charles Vandevelde was superintendent.

Asphalt Pavement Costs with Municipal Plant

By John W. Cunningham*

The City of Spokane, Wash., has for the past six years owned and operated a municipal asphalt-paving plant. While the primary use of this plant is for repair and maintenance work and the policy of the city has not been to eliminate entirely the contract system of doing paving, a number of jobs have been done by the city forces, and thorough cost records have been kept. This work has in general shown a substantial saving under contract work. It has also given the city engineering department under Morton Macartney, City Engineer, a close idea of the cost of asphaltic pavements under local conditions and a more logical basis for engineering estimates of street-improvement work.

The municipal plant has also been a means of reducing the cost of paving through selling the asphaltic mixture, or "hot stuff," to contractors at nearly a cost price. The asphaltic mixture is delivered to the contractors' wagons at the plant. Under this method the contractor is relieved of owning the most expensive and least flexible part of his equipment, which enables smaller contractors having less overhead expense to bid upon work and has, in the case of Spokane, resulted in a substantial reduction in costs.

The Department of Public Works of Spokane has adopted as standard an asphaltic concrete pavement conforming closely to the Topeka specifications and known locally as "Spokane bituminous." A considerable amount of this pavement has been laid with very satisfactory results, either as a topping upon a coarser mixed asphaltic base or upon a concrete base.

The accompanying detailed costs are for the "Spokane bituminous" pavement laid by the city forces. The first tabulation is for pavements of 5-in. total thickness, made up of a 2-in. asphaltic concrete top on a 3-in. asphaltic base and requiring two spreadings, rakings and rollings. The second tabulation is for 2 in. of asphaltic concrete laid upon a concrete base, the cost of base not included. It will be noticed that the costs, $0.88 and $0.41 respectively, show a rough relation to the thickness. Another piece of pavement upon which costs are available is 3 in. thick, laid upon a concrete base. The cost of laying this was $0.62 per sq.yd., or about 50% more than for the 2-in. pavement.

In all asphaltic pavements the most expensive single item will usually be the asphaltic cement. Since the other materials and the plant and labor costs will vary but slightly with a change in the mixture, a general rule may be stated that the coarser the mixture the less asphalt will be required and hence the less the total cost. Under similar conditions a fine mix, such as a sheet asphalt, would be more expensive than the "Spokane bituminous" pavement, and the coarser mixed asphaltic concretes would be less expensive.

TABLE 1. COST OF A 5-IN. ASPHALTIC PAVEMENT (2-IN. ASPHALTIC CONCRETE TOP ON A 3-IN. ASPHALTIC CONCRETE BASE), CITY OF SPOKANE, WASH.*

Plant Cost		
Asphalt top, 233.34 tons @ $18-$22	$4,309.15	$0.1605
Asphalt base, 177.92 tons @ $18-$22	3,274.15	.1220
Aggregate top, 1,779 cu.yd. @ $1.25	2,223.75	.0828
Aggregate base, 2,324 cu.yd. @ $1.25	2,905 00	.1082
Fuel, power and supplies at plant	1,120.00	.0417
Labor	2,462.70	.0917
Total cost out of plant	$16,294.75	$0.6069
Haul by teams and motor trucks, length of haul not available	2,563.50	.0955
Street Cost		
Labor, laying pavement	2,744.30	.1022
Total of plant cost, haul and street cost	$21,602.55	$0.8046
Plus 10% overhead and depreciation		.0805
Grand total cost per sq.yd.		$0.8851

*Average of two jobs; total yardage, 26,847.

TABLE 2. COST OF 2-IN. ASPHALTIC CONCRETE TOP LAID ON A CONCRETE BASE BY THE CITY OF SPOKANE, WASH.*

Plant Cost		
Asphalt, 401.012 tons @ $16-$16.50	$6,491.50	$0.1418
Sand, 2,594 cu.yd. @ $1	2,594.00	.0567
Rock, 777 cu.yd. @ $1.30-$1.45	1,105.95	.0242
Coal, 79.95 tons @ $6	479.70	.0105
Power	189.95	.0041
Labor	1,708.71	.0373
Total cost out of plant	$12,569.81	$0.2746
Haul from 1.25 to 2.10 mi. Both motor trucks and wagons used, costs not segregated. With wagons, snap teams were used on heavy grades. Average cost per ton-mile was $0.232	1,836.03	.0401
Street Cost		
Paint, 12.25 bbl. @ $7.50	$91.90	$0.0020
Wood, 13.25 cords @ $6	79.50	.0017
Coal, 7.95 tons @ $6	47.70	.0010
Top sand, 19 cu.yd. @ $1.375	26.15	.0006
Labor	2,436.93	.0532
Total cost of laying on street	$2,682.18	$0.0585
Total of plant cost, haul and laying per sq.yd.		$0.3732
Plus 10% overhead and depreciation		.0373
Grand total cost per sq.yd.		$0.4105

*Average of three jobs; total yardage, 45,802.

Fire-Resisting Materials for use in constructing the deck structures of excursion and passenger steamers are to be investigated by the Bureau of Standards. The Bureau has issued a circular inviting manufacturers of such materials to submit samples for investigation and test.

*Consulting Engineer, Portland, Ore.

Mass Diagram for Power and Water-Supply Computations

By W. L. Butcher*

The following notes describe the extension of the "mass" diagram to water-supply and power problems of a somewhat more complex nature than is usual with this graphical device. The simple mass diagram for the study of storage and water-supply, known as the "Rippl" diagram from the name of its originator, is too familiar to require lengthy explanation. The original method consisted in the construction of a mass curve showing the summation day by day, or month by month, of the net accumulated quantity of water resulting from the difference between the runoff and an assumed draft during the period. From such a diagram a graphical study of the required storage to maintain on an assumed draft, the length of time storage will be drawn upon and other factors may be determined.

This method was simplified by John R. Freeman, who in studies for the New York water-supply[1] adopted the idea of plotting the accumulated runoff directly and without first assuming and deducting a certain draft. This not only lessens the preliminary work, but widens the scope of the diagram. It usually occurs in water-supply problems that the storage reservoir is located at the point where the runoff is calculated, and the previously mentioned devices have been applied to this condition.

The following explains the extension of the idea to include another class of problems where it is desired to obtain the conditions of flow that may be maintained at some point downstream, due to storage reservoirs at the headwaters or at other points. Such problems arise in water-power and pollution considerations and in the matter of log-driving and other cases. A mode of treatment of these latter problems is given in Water-Supply Paper 198 of the United States Geological Survey, as developed by Walter H. Sawyer; but the procedure described later, while very simple, does not seem to have been previously used and is believed to be somewhat simpler than the method already mentioned and of wider application.

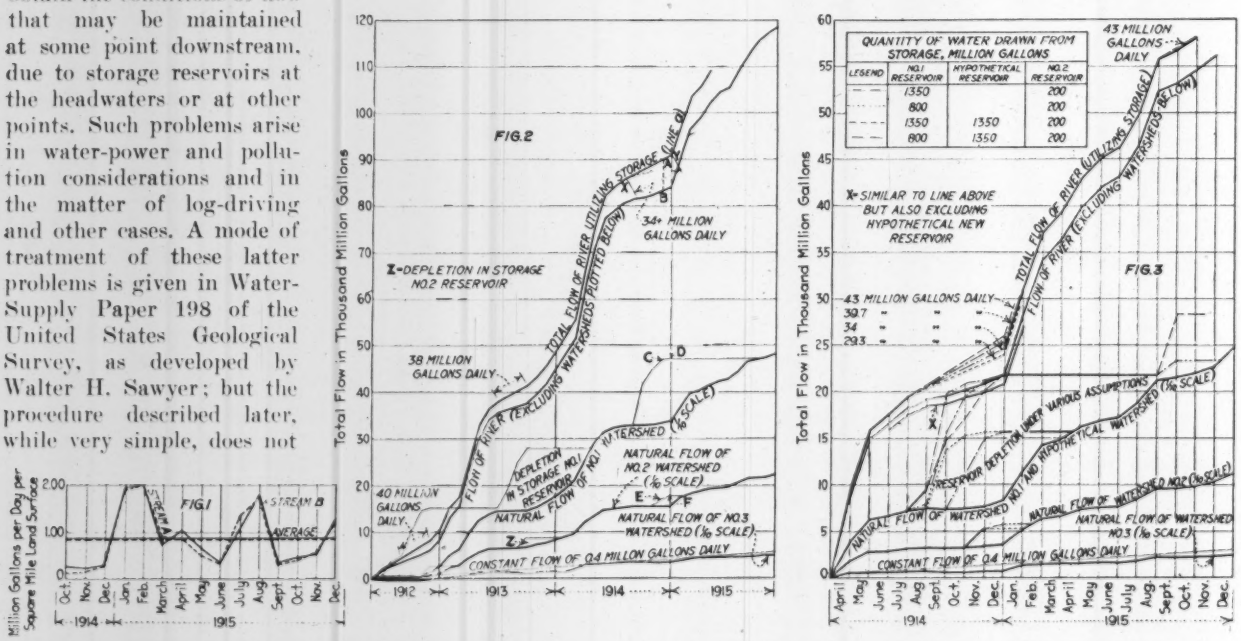


FIG.1
STREAM B
AVERAGE
Million Gallons per Day per Square Mile Land Surface
1914
1915
FIG.2
Total Flow in Thousand Million Gallons
TOTAL FLOW OF RIVER (UTILIZING STORAGE FLOW OF RIVER (EXCLUDING WATERSHEDS PLOTTED BELOW)
34+ MILLION GALLONS DAILY
Z=DEPLETION IN STORAGE NO.2 RESERVOIR
38 MILLION GALLONS DAILY
DEPLETION IN STORAGE NO.1 RESERVOIR
NATURAL FLOW OF NO.1 WATERSHED (1/6 SCALE)
NATURAL FLOW OF NO.2 WATERSHED (1/6 SCALE)
40 MILLION GALLONS DAILY
NATURAL FLOW
NATURAL FLOW OF NO.3 WATERSHED (1/6 SCALE)
CONSTANT FLOW OF 0.4 MILLION GALLONS DAILY
1912
1913
1914
1915
FIG.3
QUANTITY OF WATER DRAWN FROM STORAGE, MILLION GALLONS
LEGEND
NO.1 RESERVOIR
HYPOTHETICAL RESERVOIR
NO.2 RESERVOIR
1350 200
800 200
1350 1350 200
800 1350 200
43 MILLION GALLONS DAILY
X=SIMILAR TO LINE ABOVE BUT ALSO EXCLUDING HYPOTHETICAL NEW RESERVOIR
TOTAL FLOW OF RIVER (UTILIZING STORAGE)
FLOW OF RIVER (EXCLUDING WATERSHEDS BELOW)
RESERVOIR DEPLETION UNDER VARIOUS ASSUMPTIONS
NATURAL FLOW OF WATERSHED NO.1 AND HYPOTHETICAL WATERSHED (1/6 SCALE)
NATURAL FLOW OF WATERSHED NO.2 (1/6 SCALE)
NATURAL FLOW OF WATERSHED NO.3 (1/6 SCALE)
CONSTANT FLOW OF 0.4 MILLION GALLONS DAILY
1914
1915

FIGS. 1 TO 3. DEVELOPMENT OF MASS DIAGRAM FOR WATER-SUPPLY AND POWER COMPUTATION

Fig. 1—Diagram showing close agreement in runoff of land of two adjacent streams. Fig. 2—General mass curve for determination of critical period; reservoirs No. 1 and No. 2 have capacities of 1,350 and 200 million gallons respectively. Fig. 3—Mass diagram for studying in detail the effect of storage

This scheme consists, in brief, in the construction of a number of mass diagrams on a single sheet, each storage reservoir and its watershed being treated as an isolated case, but the sum of all of them, including the remaining portions of the watershed tributary at the point where conditions are to be studied, amounting to the total watershed under consideration.

The starting point for the calculation is chosen at a time when from an inspection of the runoff records it is seen that the reservoirs will be full and overflowing. The accumulated amounts of runoff at the end of each daily or monthly period are then determined for each of the separate portions into which the watershed has been divided, and the results are plotted as mentioned, on a single sheet.

The method may perhaps be best understood from here on by an explanation of the steps taken in an actual study. The situation was as follows: On a stream having about 100 sq.mi. of watershed it was desired to ascertain the maximum flow that could be maintained during the summer months of a dry year. On the stream at widely separated points there were two storage reservoirs that could be regulated to suit conditions. Another reservoir was used as a source of water-supply for a town on the watershed; and it was assumed that, except when the records indicated that this reservoir would be overflowing, a constant quantity equal to the average water consumption would be contributed from this source.

One year's record of runoff of the stream was in existence, also complete information as to the area of water surface and storage conditions. From this information it was possible to calculate the yield of 1 sq.mi. of land surface, which was compared with the yield per square mile of land during the same period of a stream near-by, concerning which records of runoff covering a great many years were in existence.

*With Metcalf & Eddy, 14 Beacon St., Boston, Mass.

[1]Report on New York Water-Supply, 1900.

The agreement in the runoff per square mile of land surface was remarkably close during the year in which comparable records were available, and the stream having a long-term record was adopted as indicating results that might be obtained on the stream under consideration during similar periods.

In this connection, while having no particular bearing on the matter of mass diagrams, it is interesting to notice what close agreement the runoffs from certain streams may bear to each other when as many as possible of the uncertain factors are removed. Such a result is shown in Fig. 1, which needs no comment except to say that the unusual divergence at low flows was examined and found to be due, in whole or in part, to errors in meter measurements of one of the streams at very low flows. One sees so many examples of the opposite result that the foregoing has a double interest.

Having then the long-time record in hand, it was found that there were three or four consecutive dry years during which the condition of the reservoirs could not be definitely known, without treating the period as a unit. So, for the purpose of determining the critical period, the runoff for each separate reservoir and for the remaining portion of the watershed was determined and summed up beginning with June, 1912, and running through 1915. A perusal of the runoff records left no doubt that all reservoirs would be full and the whole watershed acting as a unit on June 1, 1912. The runoff for each reservoir was determined in the usual way, by calculating the yield of the land surface and water surface and adding together for each month.

After the monthly yield from the beginning was added up, the results were plotted on a small scale for the purpose of getting a general idea of the situation and to make it possible to pick out the critical period to be afterward treated in more detail. This operation is shown in Fig. 2. On account of the small area of watershed tributary to each reservoir in comparison with the remaining portion these latter figures are plotted on a $\frac{1}{10}$ scale.

Beginning now at the left-hand corner, the separate ordinates at the end of each month are summed up and plotted directly above, forming a new curve representing the combined flow of all the separate portions. This is represented by line *A*. As already explained, the flow from Reservoir No. 3 is not the natural flow of the watershed, but is regulated to give a continuous flow of 400,000 gal. daily except when overflowing, and a "rate" line governing this condition is drawn on Reservoir No. 3 curve, according to well-known mass-curve methods, before the summing up is started.

As the season of low flows approaches, the line representing the total flow tends to become more and more nearly horizontal, representing a lower and lower rate of flow. When this condition occurs, an arbitrary line is drawn from a summit, as at *X*, having an inclination that represents a rate which it is assumed may be maintained by the use of the available storage. The vertical distance then at any point between the curve showing the flow of the river, without the subsidiary watersheds, and the line just mentioned as having been drawn measures the additional accumulated flow that must be obtained from the subsidiary watersheds in order that the assumed rate may be maintained. In the example under consideration the additional accumulated flow necessary at the section *AB* amounts to 6,800 million gallons. The natural accumulated flow from Reservoirs Nos. 1, 2 and 3 amounts to 5,250 million gallons, as measured by their respective ordinates, leaving 1,550 million gallons additional to be supplied from storage, as shown by the new curve of runoff from Reservoir No. 1 and Reservoir No. 2, plotted at *C* and *E*.

If the assumed maximum rate of flow that can be maintained is too high, the rate must be reduced by drawing a more nearly horizontal line, until the necessary accumulated flow just exhausts the storage available, as shown at *D* and *F*.

When the rate of flow from the river, without subsidiary watersheds and storage, becomes greater than the maximum flow that has been maintained with the aid of storage during the previous month, which fact is indicated by a convergence of the two lines indicating total flows, the supply from the subsidiary watersheds is cut off, as shown by the horizontal lines, and the reservoirs begin to fill. While the reservoirs are filling, the sum of the ordinates to the lines representing the accumulated flow of the subsidiary watersheds remains a constant quantity except as slightly affected in this particular case by the flow from Reservoir No. 3. This results in the two curves of total flows remaining parallel until the reservoirs are full, when they begin to diverge owing to overflows from the reservoirs. Practically, it can generally be determined by inspection at what point the greatest depletion in storage will take place. The maximum storage lines can then be drawn, the ordinates summed up, added to the flow of the main river and a maximum flow line extended backward to a point of tangency with the curve of natural flow, as *X-Y*. The rate that this line represents can then be calculated, no preliminary trials being necessary.

This small-scale diagram having been completed, it is observed that the minimum conditions occur during the late fall of 1915. With this information in hand, a new curve on a larger scale is plotted beginning Apr. 1, 1914, when, according to Fig. 2, all reservoirs will be overflowing. This curve is shown in Fig. 3 and is on a large enough scale to make a detailed study. For the purpose of representing the adaptability of the diagram the following conditions may be noted: First, when Reservoir No. 1, which has a storage capacity of 1,350 million gallons, and Reservoir No. 2, which has a storage capacity of 200 million gallons, may be completely drawn off during a dry year; under these conditions a maximum flow of 34 million gallons per day can be maintained during the dry season. Second, when owing to local conditions it is not advisable to draw more than 800 million gallons from Reservoir No. 1, 200 million gallons still being available from Reservoir No. 2; under these conditions the maximum supply that can be maintained in a dry season is 29.3 million gallons per day.

In the particular case under consideration it was desired to ascertain what increased flow might be obtained during a dry season by the addition of another storage reservoir of a capacity equal to Reservoir No. 1. As this suggestion was a tentative one it was assumed that the reservoir to be constructed would be exactly similar, as to area, capacity and contributary watershed, to Reservoir No. 1. This assumption results in a new line representing the mass curve for this tentative reservoir exactly superimposed on the mass curve of Reservoir No. 1, which fact must not be forgotten in considering the possible conditions if this new reservoir were in existence.

It also results necessarily in the construction of a new line representing the flow of the main stream, exclusive of contributary watersheds, as noted on the diagram. Following through one of the sets of conditions—that in which the two existing reservoirs can be completely drawn off—we find that in order to maintain in a year similar to 1914-15 a flow of at least 34 million gallons per day at all times the following procedure would be necessary:

During August the flow of the watershed would be less than the required amount, necessitating the addition of water from storage in Reservoir No. 1 to the amount of about 50 million gallons. During the next month the amount required is still larger; and on Nov. 1 this reservoir is completely drawn off and a further quantity is drawn from Reservoir No. 2, the gates of No. 1 still remaining open in order to utilize whatever water naturally flows from the watershed of No. 1 during this period.

During January, 1915, on account of increased flow it is seen that neither water from storage nor the natural flow from the subsidiary watersheds is necessary in order to maintain a flow of 34 million gallons per day, and all gates are closed, resulting in the line of accumulated flow from reservoirs becoming horizontal. The progress of filling may be measured at any point by the ordinate between the line of natural flow of the watershed and the horizontal line representing no outflow. It will be seen that No. 1 failed to fill entirely during the spring of 1915, that a slight draft was again required during September, 1915, and that the reservoir did not entirely refill until December, 1915.

In this method of treatment there are three factors of more or less importance, which are not taken into account: First, the changing area of water surface as the reservoirs are drawn down. This factor is not considered, in view of indeterminate features of such an investigation, as of great importance. Second, losses by evaporation or otherwise in the passage of water from the reservoirs to the point under consideration. There are no available data on this point. In the particular investigation under consideration gagings at the outlet of the reservoir and a point several miles downstream showed no losses from this cause. Third, no account is taken of the time necessary for the passage of water from the reservoirs to the place under consideration. Except in the very largest drainage areas no appreciable error would be due to neglect of this factor

Design of a Water Brake*

By Winslow H. Herschel†

The writer was recently called upon to design a 500-kv.-a. 100-per cent. power factor, or 745 b.hp., turbine with 50-per cent. overload capacity. The maximum load to be measured on test was somewhat over 1,100 b.hp. As no generator was available, it was necessary to design a dynamometer of sufficient capacity to absorb this load.

When it is a question of small amounts of power and slow speeds, a friction brake may be employed in which the load is applied by brake shoes, ropes or bands bearing on the circumference of a wheel. If the heat developed is considerable, the wheel must be water-cooled, but this becomes impracticable for very large loads. The fan dynamometer is also useful for small powers and has been used to some extent for testing automobile engines.

Not every dynamometer that will measure large amounts of power is suitable for use with steam turbines. The torsion dynamometer could not be used, because it measures but does not absorb the power, though this type of transmission dynamometer is well-adapted for use on steamers where a long propeller shaft is available. The Alden dynamometer will also measure very large powers, but not at the high speeds used with steam turbines. A cast-iron disk is mounted on the shaft and is surrounded by a casing to which are attached copper plates that lie one on either side of the disk. The friction between the copper plates and the disk is varied by the pressure of the water admitted to the casing. Since the disk must be of cast iron, the peripheral speed is limited to about 7,000 ft. per min. The power is limited by the heat that can pass through the copper plates, so that the large-capacity brakes require more than one disk.

The simplest water brake, the only type of dynamometer left to consider, consists of one or more smooth disks mounted on a shaft direct-connected to the turbine shaft and inclosed in a casing. In our brake the tendency of the casing to revolve was resisted by a lever fastened to the casing and acting on an ordinary platform scales, in place of the spring used in Froude's dynamometer. For convenience in calculating the horsepower the length of lever from the center of the water-brake shaft to the point where the lever acted on the scales was made 5 ft. 3 in., so that the power could be calculated by the formula,

$$B.hp. = \frac{W(2\pi R)n}{33{,}000} = \frac{Wn}{1{,}000} \qquad (1)$$

where

W = Reading of platform scales, in lb.;
n = Revolutions per minute, of turbine and brake, from tachometer;
R = Length of lever.

In any water brake the water must be supplied to the space between the rotor and the casing in sufficient quantity to prevent the formation of steam in any considerable amount. The required rate of feed of water may be calculated from the formula,

$$Q = \frac{b.hp.(33{,}000)}{778(t_2 - t_1)8.32} \qquad (2)$$

where

Q = Rate of feed of water, in U. S. gallons per min.;
t_1 = Temperature of water at inlet, in degrees F.;
t_2 = Temperature of water at outlet, in degrees F.;
778 = The mechanical equivalent of heat.

To find the rate of feed in our case, we must assume values for the temperatures. If we take the inlet temperature as 80 and the outlet temperature as 180, so as to avoid the boiling point of water by a safe margin, we shall have a rise of temperature of 100 deg. F. Introducing this value and that of 1,100 for the power into equation 2, we have

$$Q = \frac{1{,}100 \times 33{,}000}{778 \times 100 \times 8.32} = 56 \ gal. \ per \ min.$$

This shows that care must be taken to provide for an ample supply of water. Furthermore, since the rotor acts like a centrifugal pump, the water must be admitted to the compartment containing the rotor at a point near the axis of rotation, where the pressure is

*From "Power," July 25, 1916.
†Bureau of Standards, Washington, D. C.

low. The power absorbed is increased by throttling the water as it escapes from the casing, thus increasing the width of the ring of water that is thrown to the circumference. The limit of power is reached when the casing is full of water and the rate of flow is not sufficient to prevent the formation of steam.

The water brake designed to meet these requirements consisted of six smooth disks 24 in. in diameter, such as are used for cutting slate. Each rotated in a separate compartment, with a ½-in. clearance between the sides of the disk and the walls of the compartment. Between each pair of disk compartments was an annular space 1¼ in. wide through which water was admitted, entering through the narrow clearance between the side walls of the compartments and the hubs of the disks. The casing was supported on roller bearings so as to be free to move with a change of load. To equalize the supply to the annular cells, the water was pumped into a tank placed directly above the casing and supported on it, and there was a pipe connection from this tank to each cell.

Froude stated in his paper that of two dynamometers of different size, "their respective moments of reaction, with the same speed of rotation in each, should be as the fifth powers of their respective dimensions." This rule would, however, not suffice to calculate the dimensions of a dynamometer of a new type. The complete formula, equation 3, is given by Roetscher.

$$B.hp. = \frac{D^{5.366} n^{2.66}}{4.704 \times 10^{16}} \qquad (3)$$

where

$B.hp.$ = Maximum power which can be absorbed by one disk;
D = Diameter of disk in centimeters;
n = Revolutions per minute.

This formula was derived from tests made with a brake having perforated disks, while our disks were without perforations, so that it seemed possible that equation 3 would give a power somewhat in excess of what could be obtained with our water brake. If we take D equal to 24 in., or 61 centimeters, and n equal to 3,600, the normal speed of our turbine, the equation gives 230 hp. for one disk, or 1,380 b.hp. for the six disks. The excess over 1,110 b.hp. seemed sufficient to allow for any lack of power of the brake or superabundance of power of the turbine.

The centrifugal pump employed to circulate the condensing water as well as to raise water to the water-brake tank was supposed to run at 410 r.p.m., but owing to a leaky suction pipe and lack of a foot valve it was found necessary to increase the speed to 540 r.p.m. The discharge pipe from the condenser was throttled from 8 down to 4 in. in diameter at the end where it discharged into the open air. After these changes had been made, it proved possible to raise the water through a 2½-in. pipe to the water-brake tank.

The absence of a foot valve and the impracticability of applying one below low tide caused considerable trouble before a method was found for starting the centrifugal pump. A piston pump belonging to a steam-engine condenser was connected with the suction pipe of the centrifugal pump at a point a few feet nearer the intake. Futile attempts were made to start the flow in the suction pipe by means of the piston pump and then keep the water flowing by means of the centrifugal pump after the piston pump was stopped. An ejector proved successful as a means for starting the centrifugal pump, but it was discarded as too wasteful of steam. The final solution was to place a flap over the 4-in. end of the discharge pipe, already mentioned, and exhaust the air through a pipe attached at one end to the priming hole of the centrifugal pump and at the other to the exhaust pipe of the turbine.

The supply of water to the water brake was, to the end, somewhat unreliable, owing to the increase in suction head at low tide. When at times the supply would entirely fail, the water in the water brake would turn to steam, and when the turbine was run without a governor, the consequent racing was a real source of danger. When the tachometer showed a rapid increase of speed and the needle appeared to be about to start around on a second lap, we knew that there was no time to be lost in closing the throttle.

Considerable trouble was experienced with the water-brake thrust bearings, which were replaced first by ball bearings and finally by roller bearings similar to those that had given satisfaction on the turbine from the beginning. Otherwise the water brake gave complete satisfaction. Trouble was experienced at times in trying to weigh large loads at low speeds, but as long as the normal speed of 3,600 r.p.m. was maintained there was no difficulty, although the load varied from 49 to 1,326 b.hp. It will be noticed that the maximum power measured was very close to the calculated maximum capacity of the brake, 1,380 b.hp. No measurements were made of the temperature or amount of cooling water used, but it is safe to say that a considerable saving was effected by using salt water instead of city water.

In the course of the tests the question arose as to the proper method of taking observations. It is of course impossible to keep the readings of the scales and of the tachometer absolutely constant by adjusting the load on the water brake. There will be unavoidable variations in boiler pressure, and it is safe to assume that readings of steam conditions at regular intervals of time will give the best average values. The question to be decided is whether readings for horsepower must be made at the same instant as readings for steam conditions. It is evident that readings of W and n in equation 1 must be taken simultaneously. But to take readings of speed and torque at regular intervals involves taking them during changes of speed, and it seems preferable to take readings only when the scales and tachometer are steady and the speed is normal, although this necessitates taking readings at irregular intervals of time. Apparently the reason why this method appears wrong to many engineers is that they assume that the consumption will vary with the speed as with the steam engine or hydraulic turbine. Yet it has been proved by theory and by test that the weight of steam that will pass through a steam turbine is independent of the speed of revolution.

The objection to taking readings at regular intervals of time is that when the speed is changing, the horsepower indicated would be too small by the amount expended in accelerating the rotor on a rising speed, and too large by the amount given out by the rotor on a falling speed. Although the few tests made by both methods failed to indicate any substantial difference in result, the method of taking readings at regular intervals was abandoned on account of being less convenient, as well as having no theoretical advantage.

News of the Engineering World

Tunnel Heading Cleared After Four Weeks' Work

Recovery on Aug. 20 of four bodies, the last of nine killed in the crib heading of the Cleveland water intake tunnel on July 24, 1916, closes the account of the serious accident. Excavation of the muck in the heading started a few days after the accident, and after the recovery of the bodies of ten rescuers who were killed by gas poisoning when they entered the tunnel. The muck filled the heading for about 275 ft. back from the face. Work had to proceed very carefully, as concrete lining blocks were found in the mass.

On Aug. 5, after about 35 ft., two bodies were found. No more were recovered until within 30 ft. of the face; on Aug. 18 two were found, and the other five were taken out during the next two days. One of the last two, lying against the shield, was S. H. Vokes, the civil engineer who was serving as acting foreman of the shift.

Evidences found in the tunnel prove that the accident of July 24 was a powerful explosion. Some of the bodies found near the face exhibited burns. Violent disturbance of muck cars, rails, and the concrete block lining showed the action of great force. As to conditions at the face itself nothing could be learned from the water-department officials.

Present intentions in the water department are to abandon the heading for some distance back of the face. There has been talk of carrying on all further work from the land heading, which is now about 2,000 ft. from junction. This, however, would delay completion of the tunnel by several months, and correspondingly postpone the date of putting into service the new West Side filter plant, to be supplied with raw water through the new intake.

On account of the divergence of the land heading from true line (noted in an item in another column), and the desirability of bypassing the present face of the crib heading with its gas streak, it has been proposed to bring the land heading into the side of the crib heading, making this junction well back of the present face of the crib heading. If this plan is followed the work can be carried on from both headings.

⁂

A Million Dollars for Roads in the Forest States

Secretary Houston of the United States Department of Agriculture has made the tentative allotments of the $1,000,000 fund for Federal aid in road building to those states containing national forests. This fund is separate and distinct from those created by the Federal-aid road act, noted in *Engineering News*, Aug. 3, p. 238.

The method followed in apportioning the money was as follows: Ten per cent. of the amount available each year will be withheld as a contingent fund. One-half of the remainder will be allotted to the states in amounts which will be based on the area of national forest lands in each state. The other half will be apportioned on a basis of estimated value of timber and forage resources which the forests contain. Amounts apportioned but unexpended within three years, and any balance of the contingent fund which remains unapportioned at the end of each year, will be reapportioned on the same basis as the original allotments.

The tentative allotments to the principal national forest states and territories are as follows: Alaska, $46,280; Arizona, $59,795; Arkansas, $11,294; California, $140,763; Colorado, $62,335; Idaho, $108,010; Montana, $69,901; Nevada, $19,195; New Mexico, $42,622; Oregon, $127,794; South Dakota, $8,115; Utah, $40,982; Washington, $91,739; Wyoming, $40,566. In addition, a total of $9,552 has been tentatively assigned to Florida, Michigan, Minnesota, Nebraska, North Dakota and Oklahoma, while the Eastern States—Maine, New Hampshire, Virginia, West Virginia, North Carolina, South Carolina, Tennessee and Georgia—in which the government is purchasing lands for national forests, have been allotted a total of $21,057.

In general states and counties must be prepared to furnish coöperation equal to at least 50% of the estimated cost of surveys and construction. This coöperation may be in the form of money, labor and materials, or the construction of roads. Where other factors are equal, the approval of a project may be determined upon the relative amount of coöperation offered.

⁂

Activated-Sludge Plant for Hermosa Beach, Calif.

The first Pacific Coast city to project an activated-sludge plant for the treatment of its sewage is Hermosa Beach, Calif. C. R. Sumner states that on Aug. 1, 1916, the trustees of that city authorized the construction of a "Brosius system of activated and aërated-sludge" treatment, a sewage-ejector plant, and an ocean outfall, and that it is expected bids will be received on Oct. 15, 1916.

The activated-sludge plant will consist of eight concrete tanks, each 12½ ft. in diameter and 13 ft. deep, an electric motor and a high-pressure blower.

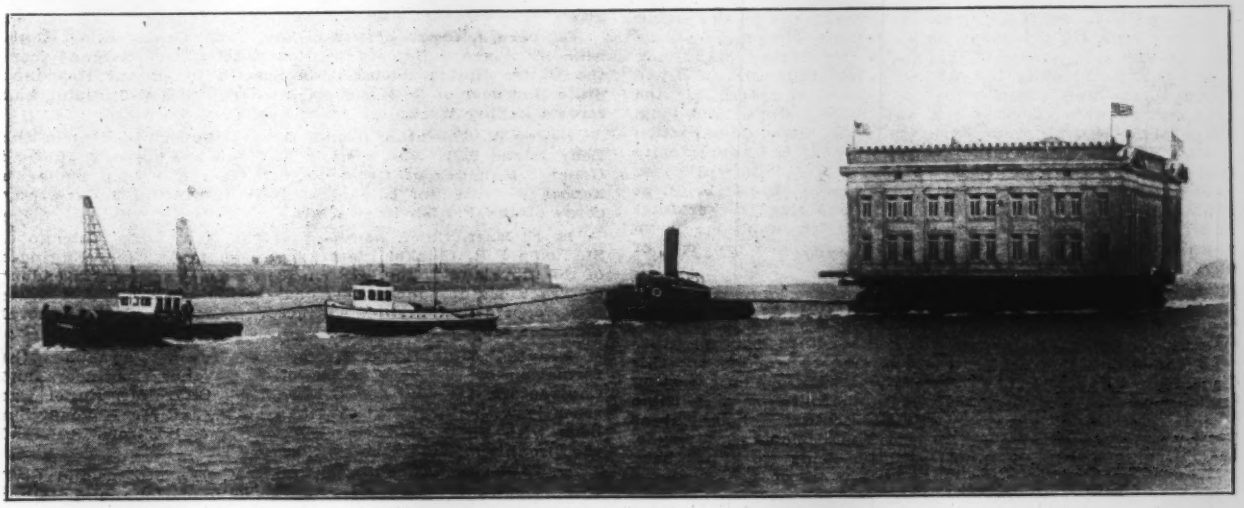
There will be two sewage ejectors, each with a capacity of 100 gal. per min., operated by electric motor-driven air compressors. These ejectors will force sewage through an 8-in. cast-iron force main 1,800 ft. long against a static head of 43 ft. to the treatment plant. From the plant there will be an ocean outfall consisting first of 830 ft. of 12-in. vitrified pipe, then 1,380 ft. of 12-in. reinforced-concrete pipe extending to the shore line, and beyond this 1,300 ft. of 10-in. cast-iron pipe laid on the ocean bed.

The force main for the 700 ft. length nearest to the treatment plant and the first 830 ft. of the outfall sewer will be laid in a 5½x7-ft. tunnel lined with timber. The estimated cost of the improvement described is $27,000.

Exposition Building Takes a Twenty-Five Mile Cruise

On Aug. 16 the Ohio State Building of the Panama-Pacific Exposition was placed on two barges and started on a 25-mi. cruise down San Francisco Bay.

The structure is a wood frame, stucco covered building, 131.5 ft. long, 80 ft. wide and 43 ft. high. It was placed at low tide on a skidway built out over the bay, and two 500-ton barges lashed together were floated into position underneath it. When the tide rose one of the Exposition's largest State buildings was a temporary ark of gigantic dimensions. It is being floated to San Carlos, San Mateo County, where it will be used as a clubhouse for the Peninsula Country Club.



TOWING THE OHIO STATE BUILDING ACROSS THE BAY FROM PANAMA-PACIFIC EXPOSITION

F. S. Buckman superintended the loading of the building on the barges. Three tugs moved it through the water at the rate of 7 mi. per hr. Captain R. P. Hurlburt was in charge of the navigation.

State Road Officials Confer

On Aug. 16 representatives of the highway departments of 37 states and of the governors of states without highway departments met at Washington at the invitation of Director L. W. Page of the Office of Public Roads and Rural Engineering to consult with him regarding the administration of the new Federal law for coöperative road building by the nation and the states. The law requires the Secretary of Agriculture to draw up regulations under which Federal money is spent by the states on their roads. Secretary Houston explained the purposes and significance of the law in an address which was heartly applauded by the delegates. Director Page stated that the fundamental object of Congress was not to distribute Federal funds to the states, but rather to help the latter to organize efficient state highway departments so that the road work in each would be conducted in the most economical manner. A set of tentative regulations was read and discussed, and the report of the discussion will be used by Secretary Houston in drafting the approved regulations. At the close of the meeting, the delegates adopted a resolution expressing their appreciation of the action of the Department of Agriculture in inviting the states to assist in this important feature of the Federal-aid work, a feature in which the law gives complete authority to the Secretary of Agriculture, as a matter of fact.

Still Few Data on Rainfall in West Virginia Mountains

Up to Aug. 21 the United States Weather Bureau had few data on the rainfall in West Virginia from Aug. 8 to 10 further than those included in the meager early reports noted in *Engineering News* of last week. A cooperative observer at Madison, Boone County, a short distance from the watershed of Coal Creek, reported a rainfall of about 6 in. on that watershed from Aug. 9 to 17.

The later reports to the United States Geological Survey are only slightly more informing. The gage keeper at McCorkle, W. Va., reports that the flood which did so much damage in Coal River Valley only slightly injured the gaging station at that point. The stay wire was caught by drift and taken out; the cable was also caught by drift but it did not break and he thinks it was not injured. Gage was reported uninjured. The highest point of the rise was 6 ft. over the top of the gage, which had been placed at the previous high-water mark. At Fuqua, W. Va., which is below the confluence of the Coal and Little Coal Rivers, the gage readings jumped from 3.73 at 7:30 a.m., Aug. 9 to +30 at 5 p.m. of that day, which latter reading was 4 or 5 ft. higher than the record.

An Investigation of Garbage Legislation has been ordered by the State Board of Health of Kansas. S. J. Crumbine, Secretary; C. A. Haskins, Engineer, and the Attorney of the Board will study the subject and prepare for submission to the next Legislature a bill designed to give the State Board of Health control of garbage disposal.

The International Nickel Co. has recently filed incorporation papers at Ottawa, which is to have $5,000,000 capital stock, and is to have its headquarters in Canada. This is the first step taken by the International Nickel Co. to carry out its agreement with the Dominion Government for the establishment of a nickel refining plant in Canada.

A New City with a prospective population of 25,000 in the near future is to be built at Sparrows Point, Md., just outside

of Baltimore, in connection with the many changes and enlargements contemplated there by the Bethlehem Steel Co. interests, which have recently acquired the property of the Maryland Steel Co. As a preliminary to the enlarged city a new sewerage system and disposal plant is to be designed and built under the direction of Ezra B. Whitman, of the firm of Norton Bird & Whitman, Chicago and Baltimore.

A Tunnel Far Out of Line and Grade—The Cleveland water intake tunnel is said to diverge 165 or 170 ft. from true line, and to be 10 ft. above true grade. These figures apply to the land heading, now over 13,000 ft. long from its starting point at Crib 3, 7,800 ft. out from shore. The divergence from line is not so important with respect to the use of the tunnel as the divergence from grade, since the latter will result in making a summit in the tunnel. The heading driven landward from Crib 5 started at the designed elevation of the intake end, and will therefore have to rise in order to join with the other heading.

A Long Sea Wall is to be built by the City Commission, Longport, N. J., which is at the southwest end of the island in which Atlantic City is located. Taking advantage of the law passed last year by the State Legislature permitting municipalities to build sea walls for the protection of beach fronts, plans and specifications are to be prepared for the construction of a concrete sea wall more than a mile long. These plans will be prepared by the Case Coast Construction Co., of New York City. It is estimated that the approximate cost of the sea wall will be $275,000. The wall will cover almost the entire beach front of Longport. It will begin at a point on the neck of the island in Great Egg Harbor Inlet and run for a distance of more than 6,000 ft. along the ocean front of Longport, covering all of the improved property of that municipality.

Courses of Instruction for Concrete Foremen and Contractors are to be started by Wentworth Institute in Boston. The work is for boys and young men and is given with the idea of training them for a future along these lines. The institute has set aside one of its best laboratory rooms for this concrete course and is fitting it up with modern testing machinery and other necessary equipment to make possible various determinations that will illustrate the technical as well as the practical sides of the applications of concrete. The first sessions will begin Sept. 18. This work has been made possible largely by the coöperation of the Portland Cement Association, the work having been planned by its Extension Division. J. C. Donaldson, Field Engineer of the Extension Division, will be located at Wentworth Institute for some time, assisting in the necessary preliminaries and afterward in starting and directing the course along the lines planned.

Calumet Harbor Development at Chicago is in its initial legislative stage. The city proposes to acquire the Lake Calumet riparian rights from shore owners before undertaking to determine the nature of development, and tentative contracts for the acquisition of these rights have been negotiated. Ordinances based upon these contracts have been drafted, but before further action is taken it is the intention of the Council Committee on Harbors, Wharves and Bridges to hold meetings for the discussion of the contracts and ordinances with the shore owners. These meetings will be held in the fall. One plan proposed provides for a central basin about 3,000 ft. wide and 2 mi. long (north and south), with an area of 536 acres and 6 mi. of waterfront. The south end of this would be connected with the Calumet River by a channel 300 ft. wide and 2,000 ft. long, with a turning basin at its junction with the river. The harbor basin would be surrounded by city land having a width of about 2,150 ft., with an area of 1,608 acres, and around the boundary of this municipal property would be a roadway and a harbor belt railway, the latter having spur connections to the various industries and main lines.

PERSONALS

T. K. Legare, former Assistant City Engineer of Columbia, S. C., has been made City Engineer.

Robert Caldwell, for the past 14 years City Engineer of Beloit, Wis., has become City Engineer of Richland Center, Wis.

Charles M. Edwards, Resident Engineer, New York State Highway Department, has been promoted to be Division Engineer.

Charles D. Snead, recently Bridge Engineer of the State Highway Commission of Virginia, is now Bridge Engineer of the State Highway Department of Kentucky.

Donald S. Dixon has been appointed Industrial Agent of the Chicago Great Western R.R., with headquarters in the Continental and Commercial Bank Bldg., Chicago, Ill.

C. B. Sadler, Assoc. M. Am. Soc. C. E., has been made Pitometer Engineer of the Baltimore, Md., water department. He was formerly Assistant Engineer of the City Sewerage Commission.

Edmund Gelwix, Assistant Engineer of the St. Louis & San Francisco R.R., Springfield, Mo., has been appointed Valuation Engineer of the Kansas City, Clinton & Springfield Ry., Springfield, Mo.

Alberto J. Pani, a Mexican member of the international commission appointed to adjust the present differences between the United States and Mexico is a prominent civil engineer and railway manager.

Kenneth H. Osborn, Assoc. M. Am. Soc. C. E., Structural Engineer of the Cleveland, Ohio, Board of Education, has resigned to become Chief Engineer of the Osborn Engineering Co., 740 Engineers' Bldg., Cleveland.

La Vern J. Charles, M. Am. Soc. C. E., Construction Engineer of Elephant Butte dam, New Mexico, has resigned from the United States Reclamation Service to become Assistant State Engineer of New Mexico, in charge of hydrographic and stream gaging work.

Henry W. Thornton, former General Superintendent of the Long Island R.R., who went to England in February, 1914, as General Manager of Great Eastern Ry., has been made an honorary lieutenant colonel of the engineering and railway corps of the British territorials.

E. E. Hart, Chief Engineer of the New York, Chicago & St. Louis R.R. (Nickel Plate), has been made Chief Engineer of the Cleveland & Youngstown R.R. also. The Superintendent of Track Maintenance and Construction of the Nickel Plate now reports to the chief engineer, instead of to the general manager, as formerly.

Frank J. Engel and **Herman P. Hevenor** announce the formation of the firm of Engel & Hevenor, Engineers and Contractors, 220 Broadway, New York City. Mr. Engel was recently with the Boston & Albany R.R., and Mr. Hevenor has been with the New York, New Haven & Hartford R.R. and the Brooklyn Rapid Transit Co.

Ygnacio Bonillas, M. Am. Inst. M. E., Mining and Civil Engineer and Subsecretary of Communications and Public Works, City of México, is a Mexican member of the international commission appointed to adjust the present differences between the United States and Mexico. He is a graduate of Massachusetts Institute of Technology, as is also his son, Y. S. Bonillas, of the Copper Queen Consolidated Mining Co., of Bisbee, Ariz.

H. H. Braun has been appointed General Superintendent for M. P. & J. T. Davis in charge of building the government dry docks and ship yard at Quebec. He was graduated from the University of California in 1908. He was Superintendent of Construction for the Arizona Power Co., leaving to become Construction Engineer for the Mexican Light and Power Co. Later he became Chief Engineer for the Mexican division of the United States Smelting, Refining and Mining Co. In January, 1912, he went to work in Spain on the plants for the 56,000- and 36,000-kw. hydro-electric developments of the Ebro Irrigation and Power Co. Since the outbreak of the war Mr. Braun has been Engineer for E. W. Moir, Representative of the Ministry of Munitions of Great Britain, engaged in organizing the munitions supply for the British government.

A. B. Fletcher, M. Am. Soc. C. E., Highway Engineer and executive officer of the California State Highway Commission, has been granted a temporary leave of absence in order to become Chief Engineer of the U. S. Office of Public Roads and Rural Engineering. His experience as the Secretary for many years of the Massachusetts Highway Commission brought him into close touch with road work in the Eastern States. Later he was in charge of the road work of San Diego County, Calif., and then organized the state highway work of California, which he has since directed. It was believed that his wide experience in both engineering and administration would be of much assistance at the outset of the coöperative road work of the nation and the states, and his temporary appointment to the position of chief engineer is the result.

Frank Rhea, M. Am. Soc. C. E., District Engineer, Eastern District, Division of Valuation, Interstate Commerce Commission, has been appointed Special Agent of the Bureau of Foreign and Domestic Commerce, United States Department of Commerce. He will undertake an investigation of the field for American railway equipment in the Far East, Australia and South Africa. Mr. Rhea was born in 1867 and was graduated from the University of Pittsburgh in 1886. He

served an apprenticeship with the Union Switch and Signal Co., Swissvale, Penn., and after spending a year on the reconstruction of the signal system of the Pennslyvania R.R. at the Broad St. station, Philadelphia, he was appointed General Foreman of Signals of the Philadelphia Terminal division of the Pennsylvania R.R. He was subsequently with the Clamond Telephone and Manufacturing Co., of Philadelphia, and was in charge of the railway signal department of the General Electric Co. when this business was transferred to the Union Switch and Signal Co. in 1911. His later experience was as Assistant and Division Engineer of the Pennsylvania R.R.

Clarence D. Pollock and **George A. Taber,** M.'s Am. Soc. C. E., announce the formation of the firm of Pollock & Taber, Consulting Engineers, Park Row Bldg., New York City. Mr. Pollock has for some time maintained an office at this address as a consulting expert on paving and municipal work. He was formerly in charge of paving for the Boroughs of Manhattan and Brooklyn, New York City; Havana, Cuba; and San Antonio, Tex. Mr. Taber has been engaged in water-supply engineering for over 15 yr. He has successively been in charge of the designing, maintenance and construction work of the Department of Water Supply in various Boroughs of New York City, and for three years was engineer-in-charge of the main office of the Croton Aqueduct Commission, during which time several large masonry dams on the Croton watershed were constructed. He has also had extensive experience in connection with the development of water power, fire protection, street cleaning and sewage disposal, and in the valuation of private water companies for municipal and state purchase. He will retain his connection with the Polytechnic Institute of Brooklyn as consulting professor of water-supply and sewage disposal.

R. A. Lundquist, Assoc. Am. Inst. E. E., Consulting Engineer of Minneapolis, Minn., has been selected by the Bureau of Foreign and Domestic Commerce, United States Department of Commerce, as a Special Agent to study markets for electrical goods in China, India, Australia, and a number of other oriental countries and South Africa. Mr. Lundquist's work will be to ascertain the types, qualities and costs, of electrical apparatus with which American goods come into competition, as well as to look into the general opportunities for the sale of such goods. Before leaving on the trip he will spend some weeks in the principal business and manufacturing centers conferring with manufacturers, exporters and business houses on the scope of the investigation. Mr. Lundquist was graduated from the University of Minnesota in 1905 with the degree of Electrical Engineer, following which he spent six years with prominent electrical houses. Since 1911 he has been in business for himself in Minneapolis, specializing in hydro-electric and transmission-line work. He is the author of "Transmission Line Construction—Methods and Costs," and has contributed numerous articles to the technical press. He is chairman of the Minnesota section of the American Institute of Electrical Engineers.

OBITUARY

A. C. Read, President of the Read Railroad Construction Co., Philadelphia, Penn., died Aug. 18 from pneumonia. He was formerly Vice-President and General Manager of the Columbia Construction Co., of Pittsburgh. He was 46 years of age and was a graduate and trustee of Pennsylvania State College. He is survived by a widow, one son and two daughters.

Howard L. Fielding, employed by the Flinn-O'Rourke Co., Inc., contractors for the East River tunnels, New York subways, as engineer in charge of soundings, was drowned on Aug. 18 while on duty. His body has been recovered and will be returned to his home in New Paltz, N. Y. He was 26 years old and had been employed as Assistant Field Engineer on the Catskill Aqueduct, both by the State of New York and by the Degnon Contracting Co. on sections up-state, and by Booth & Flinn, Ltd., on New York City sections.

Ernest F. Tabor, M. Am. Soc. C. E., Project Manager for the United States Reclamation Service of the Flathead project, Montana, died on Aug. 20. Mr. Tabor was born in 1866 and has been engaged in engineering work for the past 30 years. For the Reclamation Service he conducted important work in a number of Western States and had charge of the Flathead project practically from its inception, including the design and construction of irrigation works costing upwards of a million and a half dollars.

ENGINEERING SOCIETIES

INTERNATIONAL ASSOCIATION OF FIRE ENGINEERS.
Aug. 29-Sept. 1. Convention in Providence, R. I. Secy., James McFall, Roanoke, Va.

ASSOCIATION OF EDISON ILLUMINATING COMPANIES.
Sept. 4-7. Convention in Hot Springs, Va. Asst. Secy., E. A. Baily, 360 Pearl St., Brooklyn, N. Y.

THE TRAVELING ENGINEERS' ASSOCIATION.
Sept. 5-8. Annual meeting in Chicago, Ill. Secy., W. O. Thompson, care General Offices, N. Y. C. R.R., Cleveland Ohio.

AMERICAN FOUNDRYMEN'S ASSOCIATION.
Week of Sept. 11. Annual meeting in Cleveland, Ohio. Secy., A. O. Backert, 12th and Chestnut St., Cleveland, Ohio.

THE AMERICAN INSTITUTE OF METALS.
Week of Sept. 11. Meeting in Cleveland, Ohio. Secy., W. M. Corse, 106 Morris Ave., Buffalo, N. Y.

NATIONAL ASSOCIATION OF STATIONARY ENGINEERS.
Sept. 11-16. Convention, Minneapolis, Minn. Secy., Fred W. Raven, 417 S. Dearborn St., Chicago.

MASTER CAR AND LOCOMOTIVE PAINTERS' ASSOCIATION.
Sept. 12. Convention in Atlantic City. Secy., A. P. Dane, Reading, Mass.

RAILWAY SIGNAL ASSOCIATION.
Sept. 12-14. Convention in Mackinac Island. Secy., C. C. Rosenburg, Myers Building, Bethlehem, Penn.

NEW ENGLAND WATER-WORKS ASSOCIATION.
Sept. 13-15. Convention in Portland, Me. Secy., Willard Kent, Narragansett Pier, R. I.

ILLUMINATING ENGINEERING SOCIETY.
Sept. 18-20. Convention in Philadelphia, Penn. Asst. Secy., C. D. Fawcett, 29 West 39th St., New York, N. Y.

ASSOCIATION OF IRON AND STEEL ELECTRICAL ENGINEERS.
Sept. 18-22. Convention in Chicago. Secy., W. O. Oschmann, Oliver Iron and Steel Co., Pittsburgh, Penn.

ROADMASTERS AND MAINTENANCE OF WAY ASSOCIATION OF AMERICA.
Sept. 19-22. Convention in New York City. Secy., P. J. McAndrews.

AMERICAN PEAT SOCIETY.
Sept. 21-23. Annual meeting in Washington, D. C. Secy., Julius Bordollo, Kingsbridge, N. Y.

AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.
Oct. 9-13. Convention in Newark, N. J. Secy., Charles C. Brown, Indianapolis, Ind.

AMERICAN ELECTRIC RAILWAY ASSOCIATION.
Oct. 9-13. Convention at Atlantic City, N. J. Secy., E. B. Burritt, 8 West 40th St., New York City.

AMERICAN RAILWAY BRIDGE AND BUILDING ASSOCIATION.
Oct. 17-19. Convention in New Orleans. Secy., C. A. Lichty, C. & N. W. Ry., Chicago.

AMERICAN GAS INSTITUTE.
Oct. 17-20. Annual meeting in Chicago. Secy., G. G. Ramsdell, New York, N. Y.

AMERICAN PUBLIC HEALTH ASSOCIATION.
Oct. 24-27. Convention in Cincinnati, Ohio. Secy., Prof. Selskar M. Gunn, Boston, Mass.

The American Chemical Society at its annual meeting, to be held in New York City, Sept. 25-30, will conduct a symposium on "Occupational Diseases."

The New York State Association of County Superintendents of Highways will hold its annual meeting at the Court House, Syracuse, N. Y., Sept. 14 and 15. There will be morning sessions only. The chairman of the executive committee is George S. Tibbits; the secretary is Dwight B. Coleman.

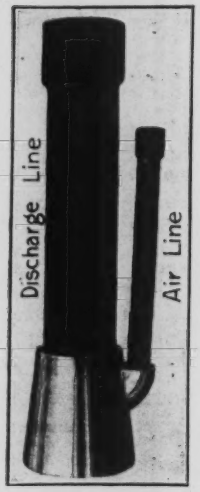
The Iowa and Illinois Sections of the American Water-Works Association will hold joint meetings Oct. 10 and 11, in Davenport, Iowa, and Moline and Rock Island, Ill. In Davenport the headquarters will be the Hotel Black Hawk. The final business session on Oct. 10 will be held at 7:30 p.m. and will be adjourned in time for the smoker at 9. The next day, at 9:30 a.m., the members of both sections will assemble at the filter plant, Rock Island Arsenal, going thence to the main pumping station of the Davenport Water Co., where lunch will be served. There will be a number of other inspection trips. Jack J. Hinman, Jr., of Iowa City, is secretary of the Iowa section.

The American Institute of Electrical Engineers, at a meeting of the Board of Directors in New York City on Aug. 8, voted to rename some of the existing technical committees as follows: Committee on Transmission to be renamed Transmission and Distribution; Railway, to Traction and Transportation; Industrial Power to Committee on Electricity for Industrial and Domestic Use; Electric Lighting to Lighting and Illumination; Protective Apparatus to Protective Devices; Electrochemical to Electrochemistry and Electrometallurgy; Records and Appraisal of Properties, to Committee on Economics of Electric Service. Upon recommendation of President Buck and the Meeting and Papers Committee, it was decided to hold the October Institute meeting, in Philadelphia, omitting the usual New York meeting. The December meeting will be held in Boston.

Appliances and Materials

Electric-Motor-Driven Diaphragm Pumps

For contracting work in large cities electric power is often found to be cheaper and more convenient than either steam or air. To meet the demand for motor-driven equipment the Domestic Engine and Pump Co., of Shippenburg, Penn., has produced a line of electric-motor-driven pumps. On account of variation in voltage requirements, make of motor, etc., prices are quoted only on the pump and mounting. Prices range from $80 to $120 for single-diaphragm pumps, to $190 for double diaphragm.



BAKER INJECTOR

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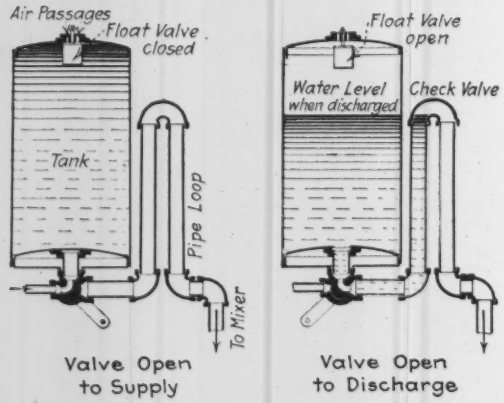
Air Lift for Injector

A foot-piece for air-lift pumping from deep wells, known as the "Baker Injector," is being made by the Chicago Pneumatic Tool Co., and is shown in the accompanying view. The air is injected into the water through a small nozzle at high velocity. It is claimed that no "booster" (vented separation chamber) is required to raise the water above the surface level, as to a tank.

* * *

Water Feed for Concrete Mixers

An automatic device for delivering water to a concrete mixer is shown in the accompanying sketch. The supply hose is attached to the 1-in. bottom pipe, where a three-way valve is fitted. When the handle is pulled down, the valve is opened to the supply pipe and water will enter the tank, a float valve being closed automatically when the tank is full. When the handle is pushed up, the three-way closes the supply connection and opens the discharge connection, allowing the water to flow from the tank to the mixer. In the discharge pipe



AUTOMATIC WATER FEEDER

is a U-shaped loop of 2-in. pipe with swivel joints, and the amount of water discharged from the tank can be adjusted or varied by shifting this loop to different angles, the amount discharged being that between the top of the tank and the top of the loop pipe. In the vertical position the amount supplied is at its minimum, but if the loop is horizontal the full contents of the tank will be discharged. This device is a specialty of the Lakewood Engineering Co., Cleveland, Ohio.

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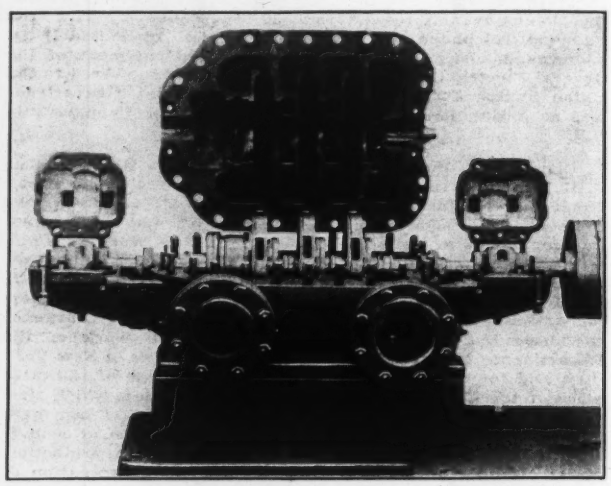
High-Speed Centrifugal Pump

A new high-speed multi-stage boiler-feed centrifugal pump is being manufactured by the Cameron Steam Pump Works, of New York City. It is designated as their "BT" type. Adequate vane length with an impeller of small external diameter has been secured by bringing the vanes well down into the hub. The shaft is protected with bronze sleeves wherever it would be in contact with the water pumped. The impellers (inclosed type) are one-piece castings. Surrounding each hub are a pair of rings, one attached to the casing and one to the impeller; these allow restoring initial tightness of stage joints without fitting. There is a hydraulic thrust-balancing device consisting of a revolving disk at the high-pressure end in a cylinder supplied with water at high pressure. Stuffing



CAMERON BOILER-FEED PUMP

boxes are deep and water sealed by lantern glands connected to the pump discharge. The bearings are self-aligning ring-oiling and horizontally split. This type is intended for steam-turbine drive.

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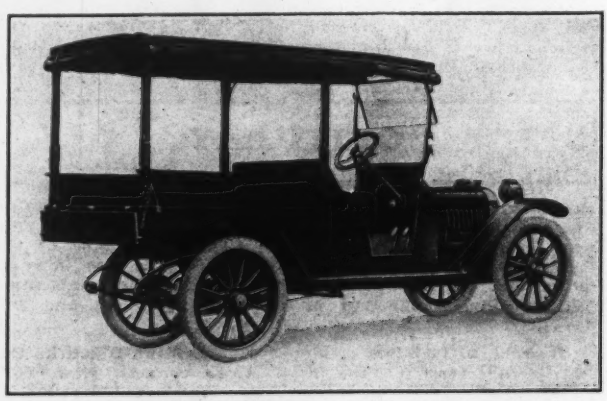
Low-Priced Motor Car

A low priced ($540) commercial motor car having a capacity of 1,000 lb. burden has been announced by the Sheridan Commercial Car Co., 1509 South Michigan Boulevard, Chicago. The motor (22 hp. at 1,800 r.p.m.) has four 3⅛x4-in. cylinders and thermo-siphon cooling system. The wheel base is 104 in.; tread, 36 in.; road clearance, 10 in.; weight, 1,500 lb. The chassis carries a sliding-gear transmission with three speeds forward and one reverse. The clutch is a multiple-disk type. The drive is through shaft and bevel gears with the pinion shaft in a housing on two roller bearings. The frame is of pressed steel channels. Forward springs are semi-elliptical; rear springs three-quarters. Wheels are 30x3 in.,



SHERIDAN 1,000-LB. CAR

of hickory and fitted with ball bearings front and roller bearings rear. Wire wheels are optional. Oversized non-skid 31x3½ in. tires are fitted. The body is of hardwood and sheet steel, 44x56 in. The top is made of matched strips covered with waterproofed cloth; side and rear curtains are of the same material. There are two acetylene headlights, two oil side lamps and one oil tail lamp.

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Gasoline Switching Locomotive

A 22-ton gasoline switch engine has been made by the Baldwin Locomotive Works for the Erie R.R. in Chicago. A four-cylinder engine gives 500 tons hauling capacity.